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Indian River Inlet: An Evaluation by the Committee on Tidal Hydraulics

by *The Committee on Tidal Hydraulics*

Prepared for Headquarters, U.S. Army Corps of Engineers

PRESENT MEMBERSHIP OF

COMMITTEE ON TIDAL HYDRAULICS

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Final report

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Preface

The U.S. Army Corps of Engineers Committee On Tidal Hydraulics held its 101st meeting in Philadelphia, PA, on 9-11 September 1992, at the invitation of LTC Kenneth H. Clow, Commander of the U.S. Army Engineer District, Philadelphia. The principal purpose of that meeting was to review the situation at Indian River Inlet, Delaware, and to assist the District in evaluating generalized channel scour there.

The Committee on Tidal Hydraulics conducted an analysis of the inlet during the period March-October 1993, and prepared this report. District liaison was provided by Messrs. Jeffrey A. Gebert and Keith D. Watson.

The Committee on Tidal Hydraulics expresses its deep gratitude to Messrs. Gebert and Watson for excellent briefings, exhaustive efforts in locating and furnishing data, and personal insights into the processes at Indian River Inlet. The Committee also thanks Messrs. Trimbak M. Parchure, Jeff Lillycrop, and Allen M. Teeter of the U.S. Army Engineer Waterways Experiment Station and Donald Raney of the University of Alabama for their prompt and helpful response to questions.

Mr. Frank A. Herrmann, Jr., is Chairman of the Committee on Tidal Hydraulics, and Mr. Samuel Powell is Headquarters, U.S. Army Corps of Engineers, Liaison.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
square feet	0.09290304	square meters
square miles	2.589998	square meters
yards	0.9144	meters

1 Introduction

Background

1. The U.S. Army Engineer District (USAED), Philadelphia, is responsible for a U.S. Army Corps of Engineers (USACE) navigation project at Indian River Inlet, Delaware (Figures 1 and 2.) Indian River Inlet has experienced progressive scour since about 1940, and the scour has accelerated since the mid 1970s. The Philadelphia District has conducted studies of the inlet scour, including a U.S. Army Engineer Waterways Experiment Station (WES) Coastal Engineering Research Center (CERC) numerical model investigation and an architect/engineer evaluation of potential remedial measures.

2. In June 1992, the Philadelphia District asked the USACE Committee on Tidal Hydraulics to review the Indian River Inlet situation and address several questions posed by the District.

Purpose

3. The purpose of this report is to answer the following questions posed by the District:

a. What factors caused the accelerated scour to begin during the mid-1970s?

(1) Does sand bypassing affect the scour?

(2) Does the tidal prism-inlet area theory apply?

b. What course of action is recommended?

(1) Is monitoring alone sufficient at present, or should planning-design-budgeting for channel stabilization begin?

(2) What is the threshold for beginning stabilization?

(3) Are the architect/engineer stabilization study findings appropriate?

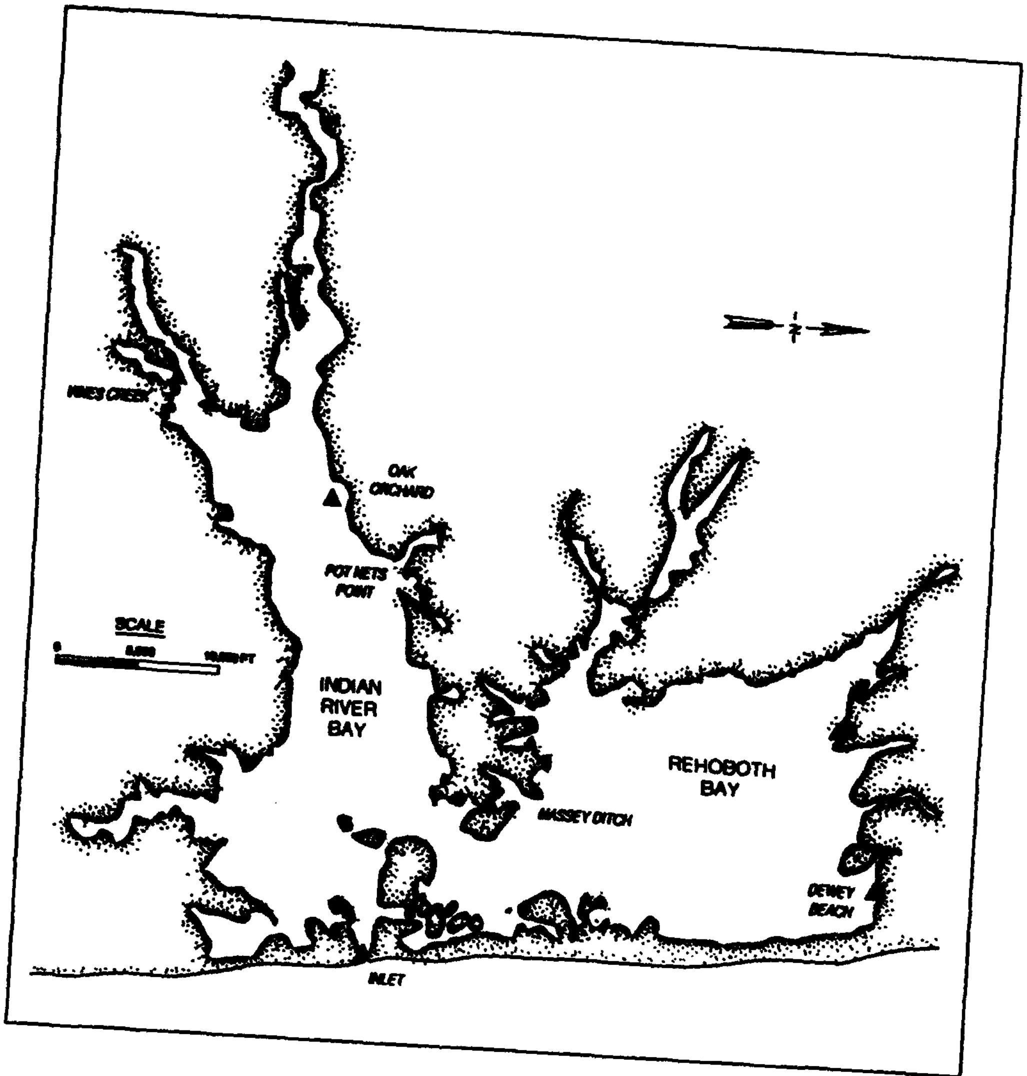

Figure 1. Site map and tide gauge locations

(4) Is local cross-section control sufficient?

(5) Should present flows be accepted and the bed armored?

c. What modeling tools and/or prototype data are needed to design a stabilization scheme?

(1) Should more hydrographic surveys be scheduled?

(2) What models are needed to evaluate the problem and design stabilization features?

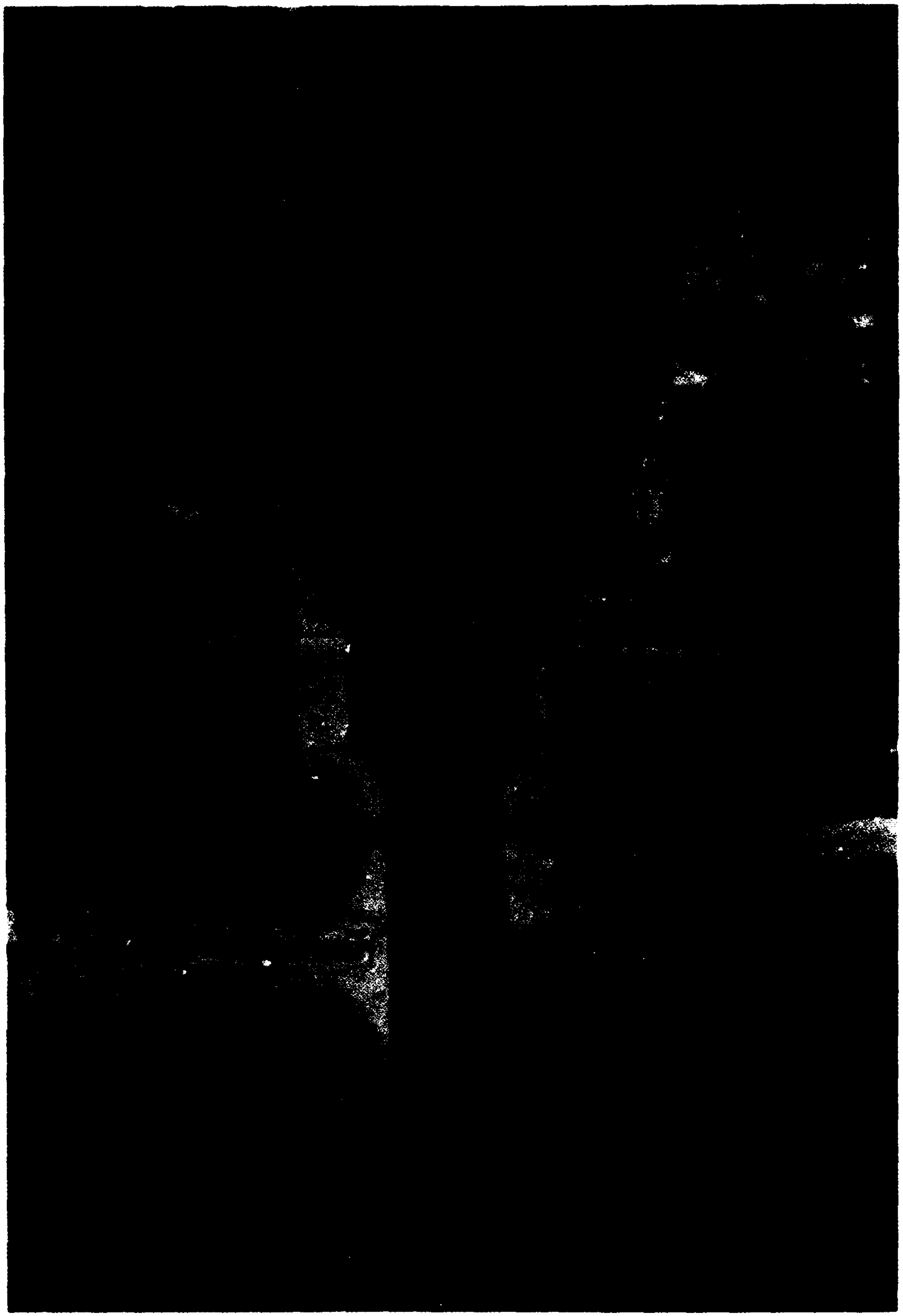

Figure 2. Indian River Inlet aerial photograph (Courtesy of Philadelphia District)

2 The Indian River Estuary Scour Problem

4. The information that follows was taken from the summary by Gebert, Watson, and Rambo (1992) and from presentations made by the Philadelphia District at the June 1992 meeting of the Committee on Tidal Hydraulics.

5. Indian River Inlet is typical of many East Coast barrier island inlets. Behind the inlet lies Indian River Bay (about 14 square miles[1] in surface area), which is connected to Rehoboth Bay (about the same size.) Figures 1-3 show the bay and inlet system at present. Prior to 1938, the inlet was ephemeral, opening in various locations after a storm, then closing again as it filled with sediment. Inlets dredged by the state also closed quickly. Between 1938 and 1940, the Corps of Engineers built parallel jetties to create a stable 500-ft-wide inlet that provided a navigation pass for recreational boats. Almost as soon as the Federal project was completed, erosion of the unprotected interior inlet shoreline began.

6. In 1941, depths throughout the inlet were less than 20 ft below mean low water (mlw). The inlet scoured both its bed and unprotected banks, increasing the average depth by about 0.5 ft per year. By 1974 some holes were deeper than 40 ft. From about 1975 on, the scour rate increased to about 1 ft per year, and in 1991 nearly all of the inlet was deeper than 40 ft with some holes over 100 ft deep. Scour of 20 to 80 ft had occurred since 1941. One exception to the overall scour is a high spot 700 ft seaward of the existing bridge, which is now remaining at a constant depth of about 35 ft or shoaling slightly.

7. About 250 ft of the seaward end of the north jetty (about 5 ft per year) has been lost to progressive storm damage. During the 1980s the inlet shoreline landward of the jetties was stabilized with revetments. Diver inspections of the jetty toes have revealed no damage from channel scour. The north jetty has been steadily losing stones from the end since the 1950s. Scour may have

[1] A table of factors for converting non-SI units of measurement to SI is presented on page vi.

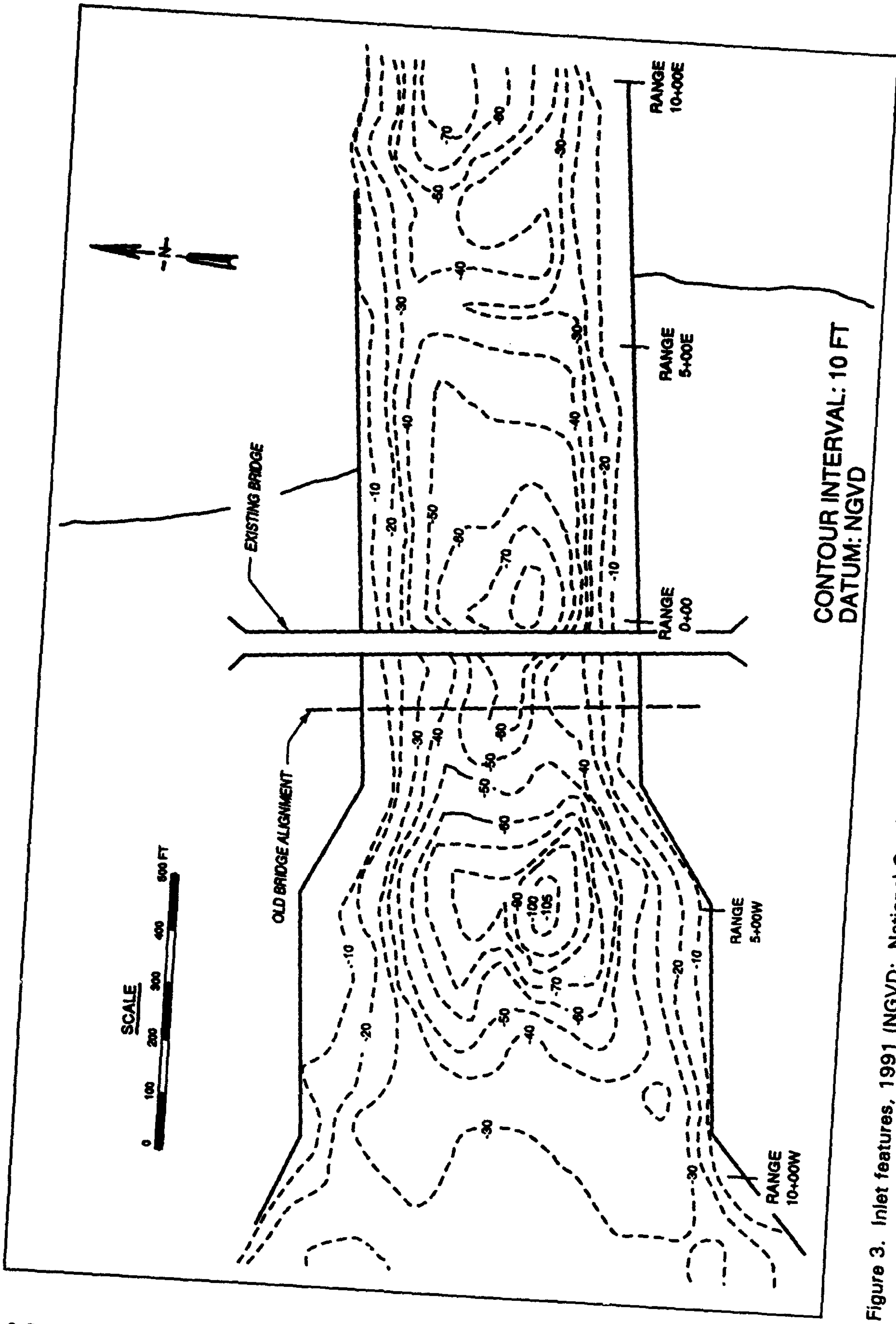

Figure 3. Inlet features, 1991 (NGVD: National Geodetic Vertical Datum)

contributed somewhat to that loss, but the primary cause has been storm damage.[1]

8. The original highway bridge, constructed on piles which significantly reduced the inlet cross section, was replaced during the late 1960s through early 1970s with a new bridge built seaward of the old. The old bridge piers were cut off, rather than removed entirely, and acoustic surveys of the inlet (Lillycrop et al. 1991) show evidence of the pile stubs and decking debris at the location of the old bridge.

9. Scour around the new bridge piers (two, located at about the one-third points across the channel) led to 1989 placement of a riprap blanket 3 to 6 ft thick and 300 ft wide from bank to bank around the piers. Design velocity for the riprap was 15 fps, and it does not seem to have sustained any significant damage since installation.

10. In 1957 nourishment of the north (downdrift) beach began, using sediments from the fringes of Rehoboth Bay. Beginning in 1972, sediment was mined from the flood tide bar of the inlet to nourish the beaches. Minimum depths over the flood shoal were nearly zero before mining began. The estimated flood shoal accumulated sediment rate was about 60,000 cu yd per year (USAED, Philadelphia 1984).

11. In 1990, beach sand bypassing was begun, passing on average about 100,000 cu yd per year from the south beach to the north. The estimated gross littoral transport is 300,000 to 500,000 cu yd per year, and the net transport is 100,000 to 150,000 cu yd per year to the north (USAED, Philadelphia 1984). (These are the best estimates available, based on calculations made using Wave Information Study (WIS) data from Station 65 north of the inlet. Confidence in the results may be reduced somewhat by the fact that computations using the nearest WIS station (Station 66) show a net littoral transport to the south, a result contradicted by all observations.) Prior to sand bypassing, the ebb shoal accumulated sediment at a rate of about 100,000 cu yd per year. Data since bypassing began are insufficient to tell if that accumulation has continued (Gebert et al. 1992).

12. A chronological history of significant inlet events is given below.

1938	14- by 200-ft Federal channel completed
1939	Jetties completed
1940	Bridge opened to replace earlier structure
1941	Sheet pile bulkhead constructed on inner shoreline
1947	Stone fill added to support pile bulkhead

[1] Personal Communication, 30 August 1993, Jeff Gebert, U.S. Army Engineer District, Philadelphia, Philadelphia, PA.

1957	Outer section of jetties repaired
1957	Artificial nourishment of north beach begun
1961	48,000 cu yd mined from flood shoal
1963	Sheet pile bulkheads repaired and extended westward
1965	New bridge opened
1972	Removal of old bridge begun
1973	Steel bulkhead walls replaced by rubble mound
	774,000 cu yd mined from flood shoal
1975	143,000 cu yd mined from flood shoal
1976	Removal of old bridge completed
1978	700,000 cu yd mined from flood shoal
1984	468,000 cu yd mined from flood shoal
1989	Stone blanket placed at bridge
1990	175,000 cu yd mined from flood shoal
	Sand bypassing begun, 100,000 cu yd per year to north

3 Inlet Processes

Sediments

13. Geotechnical investigations have shown that the barrier island medium to fine sands extend down to about -35 ft NGVD, varying from -23 ft at the jetty tips to -40 ft at the south abutment of the highway bridge. Representative grain sizes (D_{35} or 35 percent finer by weight) are 0.35 mm on the flood shoal, 0.28 mm on the ebb shoal, and 0.30 to 0.42 mm on the beaches. Below the sands are lagoonal clays and silts to a depth of about -50 to -100 ft NGVD, below which consolidated Pleistocene sands and gravels occur. Deep cores in the area are sparse, so the Pleistocene sediments may be closer to the surface in some locations.

14. The clays and silts layer is soft at the interface with the surface sands, with a split spoon sampler requiring only two to six blows per foot. Somewhat lower in the layer the sediments are slightly stiffer, requiring four to six blows per foot. Kraft and John (1976) confirm this description, reporting that the upper portion (about half) of the layer consists of soft lagoonal sediments, and the lower portion is made up of somewhat stiffer marsh deposits. Cores show some thin layers of peat (USAED, Philadelphia 1984).

Hydrography

15. Cross-sectional depth profiles have been regularly measured by the Philadelphia District. Hydrographic data used here are at inlet ranges spaced 500 ft apart from 1,000 ft east (Range 10+00 E) of the bridge to 1,000 ft west (Range 10+00 W) of the bridge. Appendix A contains a tabular presentation of the profile data. Figure 3 shows the range locations. Plates 1-5 show cross-sectional profiles of the five ranges as they have varied since 1941. Only Range 0+00 (Plate 3) exhibited a shallower maximum depth in 1992 than in 1988, because of the riprap blanket placed at the bridge in 1989.

16. Figure 3 shows the inlet depths for June 1991 in contour form along with the range numbering system used in hydrographic surveys. The inlet

exhibits a sinuous, basin-ridge topography with the following major features, described from the ocean landward.

a. A basin (maximum depths greater than 70 ft below NGVD) lies just eastward of a line connecting the ends of the jetties. The centerline of this basin is located in the northern half of the cross section.

b. A ridge (maximum depth 35 ft below NGVD) is centered about Range 6+85 E. The ridge saddle is toward the north side of the section.

c. Depths increase relatively slowly westward of the ridge, with a maximum depth of 80 ft in a basin at about Range 1+55 E. The centerline of the basin is slightly south of the center of the section.

d. A ridge (maximum depth 55 ft below NGVD) is located at about Range 1+55 W, corresponding closely to the location of the old bridge.

e. A basin with maximum depths greater than 105 ft below NGVD lies at about Range 4+00 W, south of the centerline of the section.

f. Maximum depths then decrease in the westward direction, shoaling to less than 30 ft below NGVD at about Range 11+00 W.

17. Table 1 shows the variation of the unweighted average inlet cross-sectional area from 1941 to 1992 for the five ranges shown in Figure 3. (The basic data are shown in Appendix A, Tables A1-A5.) Range 0+00 data were excluded from the average for 1992 because of the riprap placement. The steady increase in depth is obvious, but the data suggest that the rate of increase is slowing.

18. Figures 4 and 5 show the rate of deepening with plots of overall average inlet depth and maximum inlet depth from 1941 to 1992 for the 2,000-ft-long inlet. Figure 4 shows that the average inlet depth increased slowly (about 0.2 ft per year) from 1941 to 1966, then began to increase more rapidly (about 0.8 ft per year). The rate of increase seems to have slowed considerably in the 1988-1992 period, but the data are too sparse to draw a conclusion at this point. The second curve on Figure 4 shows the average depth of the central, 300-ft-wide portion of the inlet from Range 10+00 E to Range 10+00 W, which excludes the protected flanks of the inlet.

19. Figure 5 plots the maximum depth at Range 5+00 W (the deepest point throughout most of the period) and the rate of depth increase at Range 5+00 W over the period of interest. Figure 5 shows a spurt in the rate of increase in maximum depth after 1974. Unlike the average depth, maximum depth shows no signs of leveling off after 1988. It appears from Figures 4 and 5 and Plates 1-5 that the surface sand layer (bottom

Table 1
Average Inlet Cross-Sectional Area, Range 10+00E to Range 10+00W

Year	Area sq ft	Rate of Change, Average Cross-Sectional Area sq ft/year
1941	5,150	--
1950	8,300	350
1960	9,570	119
1966	11,200	306
1974	13,200	240
1978	16,900	837
1988	22,000	541
1992[1]	22,900	200

[1] Excludes Range 0+00.

elevation -35 ft NGVD) began to disappear during the late 1960s and was mostly gone by 1980.

20. Plate 6 shows the change in average depth over time for each of the five ranges. All five ranges exhibit similar patterns, with the exception that Range 5+00 W has increased in depth much more rapidly than the others since 1978. On average, the curves for the other ranges suggest a decrease in the rate of deepening after 1978.

21. Plate 7 illustrates longitudinal profiles of average depth for five of the years of record. It shows the progressive movement of deepening from the seaward end upstream, culminating in the rapid erosion of Range 5+00 W from 1974 through 1992. That sequence is consistent with scour of the sand bed beginning at the seaward end and progressing inland as the supply of sand for a hypothesized net landward transport is sharply decreased. (Paragraphs 44-47 show the net landward hypothesis to be reasonable.) Placement of the riprap blanket at the bridge (Range 0+00) in 1989 may have contributed to the 1988-1992 erosion at 5+00 W by partially blocking the sand supply that would have been contributed from the offshore area.

22. Plate 8 is a time-history plot of stations across the channel at Range 5+00 W. The stations indicate distance from the north bank in feet.

23. The rate of inlet depth changes is displayed in three ways in Table 2, as a sediment loss rate in cubic yards per year, as an average depth increase rate in feet per year, and as a maximum inlet depth increase in feet per year. All were computed from Range 10+00 E to Range 10+00 W using the range depths in Table 1, with 1992 Range 0+00 data excluded because of the riprap placement.

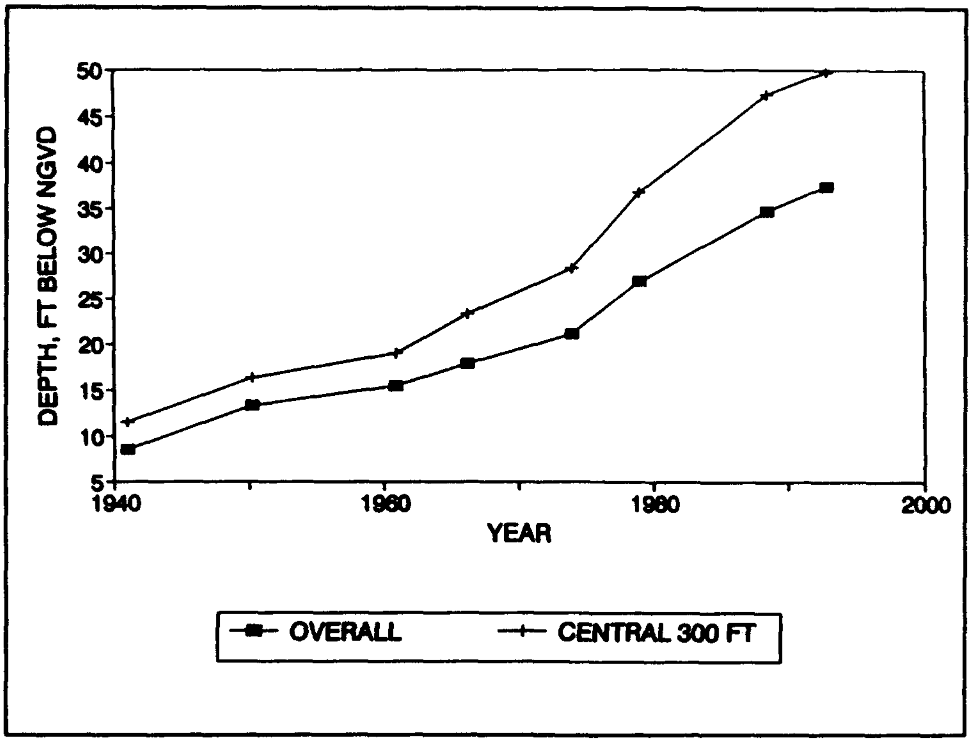

Figure 4. Average inlet depth, 1941-1992

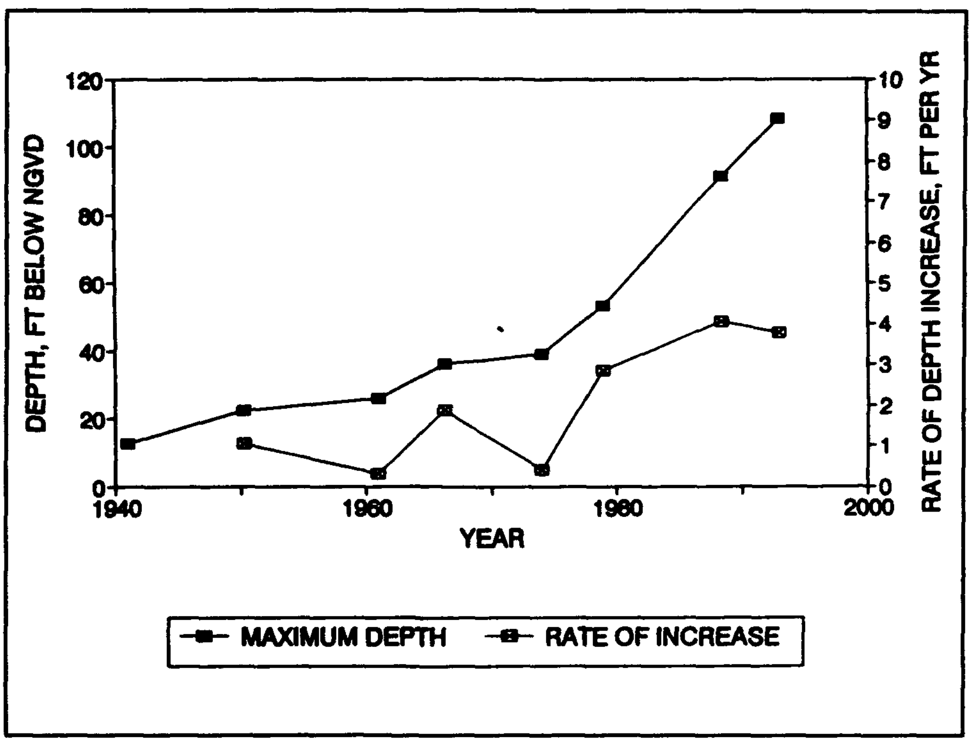

Figure 5. Maximum inlet depth changes, Range 5+00W

Table 2 Rate of Inlet Depth Change			
Period	Sediment Loss cu yd/year	Average Depth Increase ft/year	Maximum[1] Depth Increase ft/year
1941-1950	18,900	0.6	0.3
1950-1960	7,600	0.2	1.3
1960-1966	16,300	0.1	-0.4
1966-1974	15,800	0.6	0.3
1974-1978	42,900	1.2	2.7
1978-1988	30,300	0.8	4.0
1988-1992	22,700	0.6	3.8

[1] Rate of depth change of maximum depth point anywhere in inlet. Sometimes less than rate of average depth change.

24. The interior shorelines of the inlet, landward of the bridge, experienced severe erosion almost immediately after completion of the jetties. Keulegan (1967) reported that the width at old Range 40+00 (approximately new Range 25+00 W) increased from 720 ft in 1939 to 1,270 ft in 1948, while the average depth decreased from -9 ft to about -8 ft msl. In 1989 that section was more than 2,000 ft wide and had an average depth of about 20 ft, with depths farther inland very shallow, generally less than 4 ft.

Hydrodynamic Behavior

25. Table 3 shows some representative tide ranges for the Indian River Inlet and Bay system. Station locations are shown in Figure 1.

26. Figures 6 and 7 show tide measurements taken by WES in 1988 (Lillycrop et al. 1991), which show the phase shifts and amplitude changes from the ocean tide through the inlet, Indian River Bay, and Rehoboth Bay. Figure 8 shows that the inlet cross-sectional average velocity (at about Range 7+00 E) leads the inlet tide by about 1-1/2 hr, demonstrating a wave character intermediate between progressive wave behavior and standing wave behavior.

27. Raney, Doughty, and Livings (1990) performed a two-dimensional numerical model study of the ocean-inlet-bay system. They verified the WES Implicit Flooding Model (WIFM) model to 1988 tide and velocity data, then ran a series of tests with geometry from 1941 to 1988 plus several geometries with inlet size larger than that of 1988. The peak tidal discharge for the 1941 geometry was 15,000 cfs at an inlet velocity of 3 fps, and the peak 1988

Table 3 Tide Ranges at Indian River Inlet, Feet			
Location	1948	1988[1]	Estimated[2] Mean
Ocean		5.3	
Spring	4.1		4.1
Mean			3.4
Neap			2.7
Spring ranges			
Inlet at bridge	2.6	3.6	
Indian River Bay	0.93[3]	3.0[4]	
Rehoboth Bay[5]	0.47	1.5	

1 - Lillycrop et al. (1991).
2 - Estimated from nearby ocean stations and back calculations.
3 - At Oak Orchard.
4 - At Potnets.
5 - At Dewey Beach.

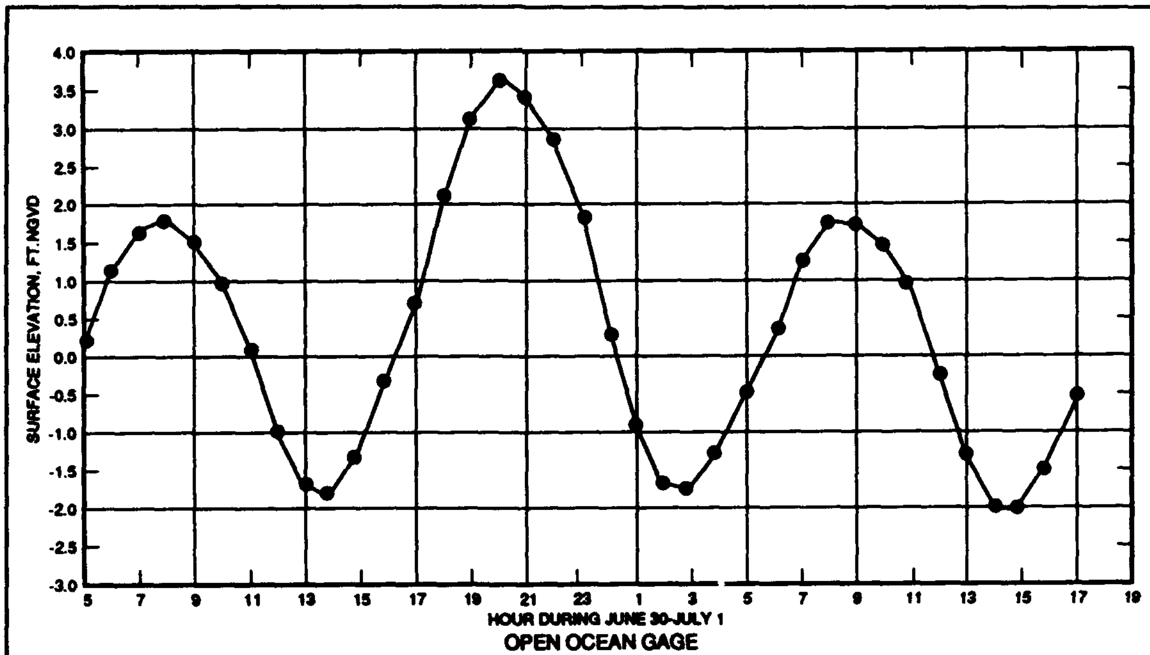

Figure 6. Ocean tide (adapted from Lillycrop et al. (1991)

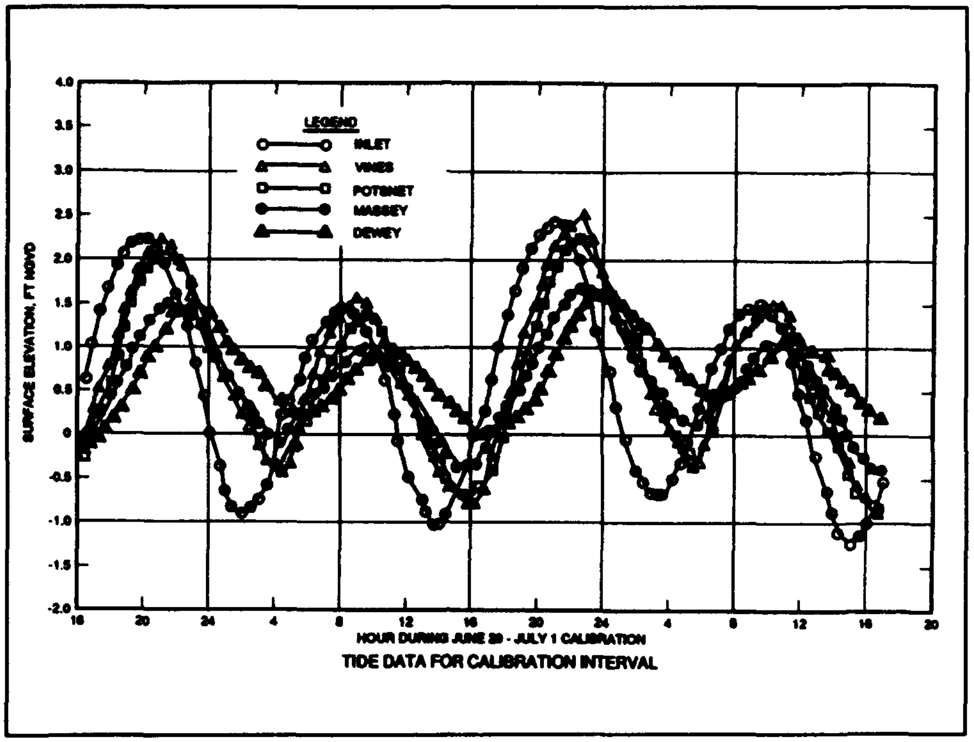

Figure 7. Measured interior tides (adapted from Lillycrop et al. (1991))

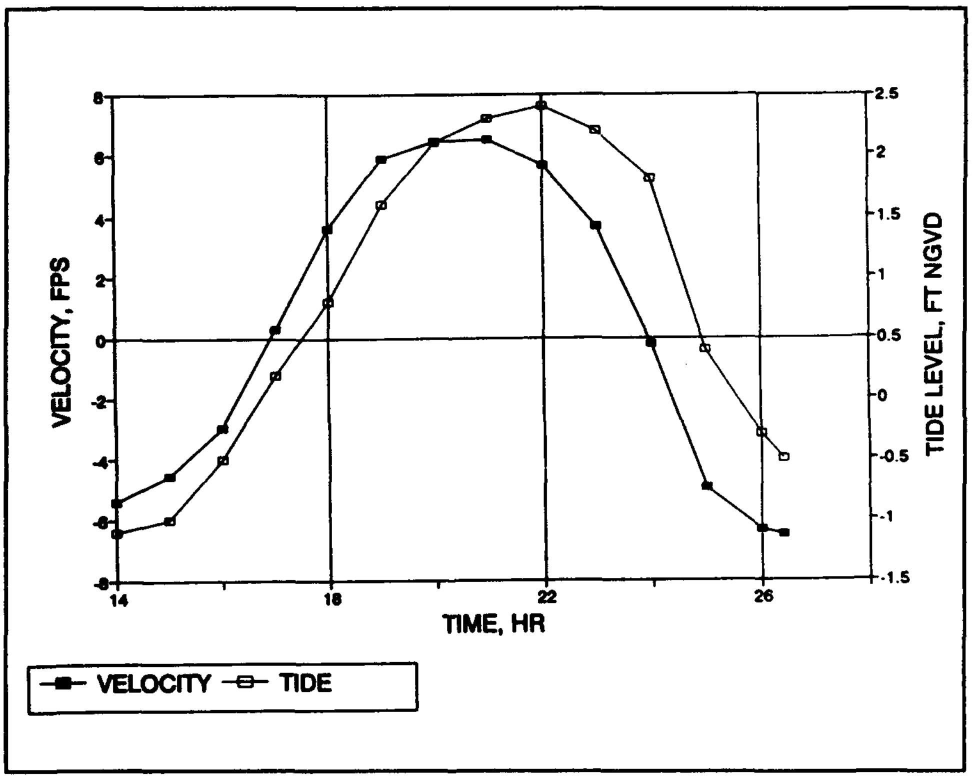

Figure 8. Average ocean tide and inlet velocity, 1988 (adapted from Lillycrop et al. (1991))

geometry discharge was over 100,000 cfs at about 7 fps. They tested inlet sizes larger than existing 1988 conditions, including cross-sectional areas of about 21,000 to 28,000 sq ft, and found that while tidal prism increased monotonically, inlet velocities peaked at about the 1988 inlet size and then declined with each increase in inlet area.

28. Keulegan (1967) calculated a composite roughness of 0.046 for the inlet for 1948 conditions, and Raney, Doughty, and Livings (1990) developed a range of cell roughnesses of 0.035 to 0.045 in verifying the WIFM model. The coefficients are not directly comparable because of the difference in one- and two-dimensional schematizations, but do indicate the most probable range of roughness coefficient.

Stable Inlet Theory

29. The Jarrett-O'Brien (Jarrett 1976) tidal prism-inlet area relationship is an order of magnitude approximation for inlets in which the sediment-scouring capacity of tidal flows is roughly in balance with the supply of sand into the inlet through longshore transport, wave suspension, and tidal flows. (This balance is sometimes expressed by the ratio of tidal prism to gross littoral transport. For example, see Bruun (1977).) This relationship cannot be used to predict an equilibrium size for Indian River Inlet because of the original serious imbalance in sediment-scouring power versus supply, and due to the sand bypassing operation which now makes its controlling conditions deviate further from that of otherwise similar inlets. Nevertheless, it is instructive to examine what the relationship says about Indian River Inlet.

30. Lillycrop et al. (1991) performed a hydrodynamic analysis using an adaptation of Keulegan's (1967) one-dimensional calculation for inlet-bay tidal hydraulics. (The latter was an effort, sponsored by the Committee on Tidal Hydraulics, that used Indian River Inlet as an example.) The results showed that the inlet could reach a stable size of about 40,000 to 80,000 sq ft when the Keulegan-King hydrodynamics curve intersected the Jarrett-O'Brien stability curve. The latter result can also be obtained by the short cut of assuming that tide ranges inside Indian River and Rehoboth Bays are both equal to the ocean tide spring range, then using that range to compute a tidal prism that is used in the Jarrett-O'Brien equation.

31. A somewhat more realistic maximum size can be estimated by calculating a tidal prism for ranges of 5.3 ft in Indian River Bay and 2.65 ft in Rehoboth Bay, based on the relationship between the two bays' tides shown in Table 3. That calculation yields a tidal prism of 3.2×10^9 cu ft and a cross-sectional area of about 62,000 sq ft. The inlet would have to have an average depth of 124 ft for that cross-sectional area.

32. Another limitation of the Jarrett-O'Brien equation for this purpose is that the scatter in the raw data is large. Using the 95-percent confidence limits on that equation and a prism of 3.2×10^9 cu ft yields a range of

probable cross-sections of about 25,000 to 140,000 sq ft, or a mean depth of 50 to 280 ft, a range which is hardly amenable to arriving at a decision on protective works.

33. O'Brien and Dean (1972) presented a useful method of plotting maximum inlet velocity versus inlet cross-sectional area for a given inlet, then noting that certain parts of the curve are inherently stable or unstable. Such a curve (see paragraph 39) was constructed for Indian River Inlet using the data from the WIFM model plus results from an application of Keulegan's (1967) tidal repletion analysis.

34. A direct extension of Keulegan's computations for the present Indian River Inlet was not possible. Keulegan's original analysis used an inlet length of 5,500 ft and an equivalent prismatic inlet area of 9,660 sq ft. Erosion of the interior shoreline has now made that a very poor approximation to the system since the defined inlet is now only about 2,000 ft long, and the rest of the former inlet is difficult to distinguish from the bay proper. Using 2,000 ft for inlet length, and a weighted average area, a Manning's n of 0.08 was found to be necessary to reproduce the observed 1988 tides of Figure 7, which is clearly not a reasonable value.

35. Although the interior portion of the former inlet is much wider now, it must still exercise some control over tides in the system through the extremely shallow flood tide shoal; therefore, that effect was included by treating it as an entrance/exit loss. Keulegan's (1967) nondimensional head loss term H was of the form

$$\Delta H = \frac{fL}{R} + m \tag{1}$$

where

f = inlet Darcy friction factor
L = inlet length
R = inlet hydraulic radius
m = entrance/exit loss coefficient, assumed = 1

Another loss term was added to the equation to account for head loss across the flood tide shoal, using

$$\Delta H = \frac{fL}{R} + \Delta H' \tag{2}$$

where

$$\Delta H' = \frac{f'L'}{R'} + m$$

and

f' = Darcy friction factor for the shoal
L' = length of shoal zone
R' = hydraulic radius of the shoal

36. The primed term for the shoal head loss is similar to that of the inlet friction term, but it was treated as a lumped head loss, using the form to compute a range of reasonable values. Darcy coefficients ranging from 0.03 to 0.07, an L' of 2,000 to 3,500 ft, and an R' of 10 to 12 ft yields a $\Delta H'$ value of 6 to 25. By trial, tide ranges were computed as described below to find a value of $\Delta H'$ that gave an Indian River Bay tide range equal to that measured in 1988. This method (ignoring Rehoboth Bay) will give a tidal prism that is slightly too low, but close enough for the present purpose.

37. For the tidal prism calculations, 2,000 ft was used for the inlet length and a uniform 500 ft for the width. For 1988 and previous years, the representative inlet cross-sectional area was based on a weighted average of the five cross sections, neglecting those portions of 5+00 W and 10+00 W that lay outside the 500-ft-wide, deep-water portion. (These computations do not require that the inlet be of constant cross section, but only that the cross section be uniform enough to permit a calculation of average flow to reasonably represent average conditions. The more nonuniform the inlet, the less accurate the results become. As discussed later, an improvement to these simple calculations requires detailed modeling.) The bay area was 4.2×10^8 sq ft. A Manning's n value of 0.035 was used for the inlet. A lumped flood shoal loss term $\Delta H' = 14$ successfully reproduced 1988 observed tides, and that value was used throughout.

38. The inlet areas and resulting tidal prisms from the Keulegan method are shown in Table 4, along with a calculated maximum inlet velocity based on an assumed sinusoidally varying velocity time-history. The sinusoidal assumption is a particularly rough one, and should be used with caution. Shown also are the maximum calculated velocities from the WIFM model, which employed a larger tide range, 5.3 ft, than the 4.1-ft average spring range employed; therefore, the velocities were adjusted by the ratio of tide ranges. The velocities are comparable, and both show that peak velocities occur for a cross-sectional area near the 1992 condition. For further inlet enlargement, the tidal prism continues to grow, but the inlet area grows faster, so maximum velocities begin to fall.

39. The data of Table 4 are presented in an O'Brien-Dean-type plot in Figure 9. Also plotted are a maximum velocity form of the Jarrett-O'Brien equation and two field measurement points from 1941 (Keulegan 1967) and 1988 (Lillycrop et al. 1991). The WIFM velocities and the 1988 field

Table 4
Bay Tidal Ranges and Inlet Velocities

Year	Equivalent Cross-Sectional Area sq ft	Tidal Range ft	Maximum Velocity Keulegan[1] fps	Maximum Velocity WIFM[2] fps
1950	6,690	0.71	3.1	3.2
1960	7,700	0.87	3.3	3.3
1966	9,200	1.13	3.6	4.9
1974	11,100	1.47	3.9	5.2
1978	14,500	2.12	4.3	5.1
1988	18,400	2.82	4.5	6.0
1992	18,800	2.88	4.5	---
--	21,000	3.21	4.5	5.9
--	23,500	3.53	4.4	5.3
--	26,000	3.75	4.2	5.0
--	28,500	3.90	4.0	4.9
--	30,000	3.97	3.9	---
--	35,000	4.07	3.4	---
--	45,000	4.10	2.7	---

[1] *Keulegan method with 4.1-ft ocean tide range.*
[2] WIFM model (Raney, Doughty, and Livings 1990) adjusted to 4.1-ft ocean tide range.

observation have been adjusted (multiplied by the tide range ratio) to make them comparable with the 4.1-ft tide range employed. In Figure 9, the Jarrett-O'Brien curve represents the line along which all Atlantic coast inlets with two jetties tend to lie, whereas the other curves represent the specific response of Indian River Inlet.

40. The most obvious fact demonstrated by Figure 9 is that the Jarrett-O'Brien curve means that stable inlet velocities for the Atlantic coast are generally between 3 and 3.5 fps, far less than the present normal spring range Indian River Inlet velocities of more than 4.5 fps. Indian River Inlet velocities have not been near that equilibrium range since the 1940s. The inlet velocity was below the Jarrett-O'Brien line in 1948, but was obviously unstable. The velocities for 1988 geometry (about 20,000-sq-ft area) are, as observed in Table 4, apparently near the maximum expected, and may even decrease with further erosion. The computed Indian River Inlet velocities are highly approximate for geometries and roughness far from the verification condition, but they do illustrate the basic situation: for inlet sizes larger than

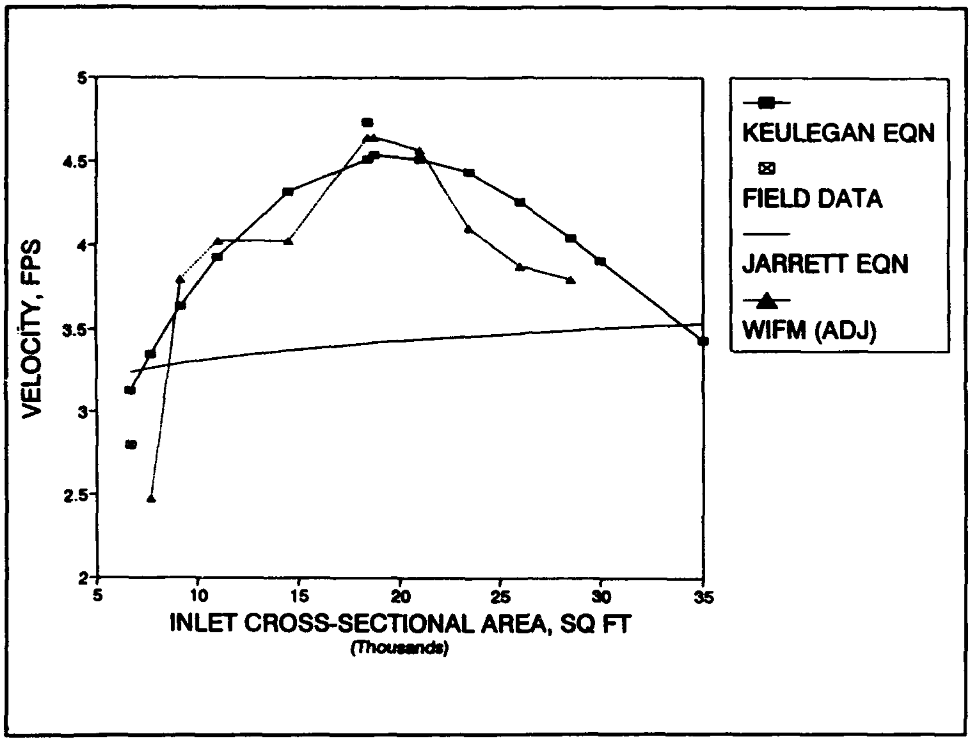

Figure 9. Maximum inlet velocity as a function of cross section

those at present, the velocities may be flat or even decreasing, but they are still far above the stability level.

41. The shape of the Keulegan method and WIFM curves illustrates an important aspect of the O'Brien-Dean curve. Points along the rising side (areas less than about 20,000 sq ft) tend to be unstable because changes in area tend to be amplified; a decrease in area reduces velocity, which further reduces sediment-scouring capacity and leads to still more inlet area reduction, and inlet area increase accomplishes the reverse. For points to the right (falling side) of 20,000 sq ft, changes in area cause compensating changes in velocity, so either erosion or infilling is opposed. These are, however, just tendencies toward stability or instability, which ultimately depends on both the hydrodynamic response shown in Figure 9 plus the supply of sand to the inlet.

42. Figure 9 could be used to support an equilibrium inlet size of about 34,000 sq ft, the point at which the Keulegan curve intersects the Jarrett-O'Brien curve; however, that cannot be made a conclusion at this point. First, the Jarrett-O'Brien curve is based on average sand supply conditions for the Atlantic coast, and it appears the Indian River Inlet supply is quite small. Second, as noted above, the data scatter in the Jarrett-O'Brien curve is large. Third, the Keulegan method velocity curve is highly approximate and does not adequately include the Rehoboth Bay response. Figure 9 definitely indicates that a stable sand bed is not expected under existing conditions, even without sand bypassing in operation.

Sand Transport

43. As a first step to identifying the stability of the inlet, a set of sand transport calculations was made to identify the size of the sand supply deficit. The calculations were performed as follows:

a. The inlet velocities measured in 1988 were smoothed such that a single, repetitive 12.42-hr tidal cycle of hourly velocities was obtained, while keeping the integral over time of the velocity and velocity squared consistent with the raw values. This provided a velocity time-history that preserved the distortion of inlet velocities by geometric effects.

b. The velocity history from *a* was scaled down to average inlet velocities by multiplying the individual values by three coefficients as shown below.

$$V_{Y,L,adj} = C_1 * C_2 * C_3 * V_{1988} \tag{3}$$

where

C_1 = the ratio of cross-sectional area where the measurements were taken (15,018 sq ft) to an average cross-sectional area $A_{Y,L}$ for the year (Y) and location (L) within the inlet needed

C_2 = the ratio of the ocean tide range for which velocities were desired (e.g., 4.1 ft) to that of the 1988 measurement period (5.3 ft)

C_3 = the Indian River Bay tide modulation factor

$$C_3 = \frac{R_Y}{R_{1988}}, \tag{4}$$

and

R = tide range in Indian River Bay for an ocean tide range of 4.1 ft (mean spring tide), and the subscript 1988 indicates the year.

The velocities were corrected with coefficient C_2 for spring, mean, and neap tide ranges as listed for the tide tables in Table 3. (Those representative tide ranges were estimated since the exact magnitudes are not critical to the purposes of the calculation.) The C_3 correction was made to account for the change in relative tide modulation between the year 1988 and both earlier and future cross-sectional areas, as calculated by the Keulegan method and presented in Table 4.

c. The adjusted velocity time-history was used to compute plane bed shear stress history by a Darcy-type equation with the friction coefficient derived from the sediment grain size via Strickler's equation, and total shear stress by Manning's equation.

d. The shear stresses computed by *c* were used with sediment characteristics and flow information to calculate hourly rates of sediment transport via the Ackers-White total load equation (Ackers and White 1973) for one 12.42-hr tidal cycle under mean, average neap, and average spring tide range conditions.

e. The hourly sediment transport rates were integrated over a tidal cycle by Simpson's rule to obtain an estimate of net and gross sediment transport rates through the inlet for each tide range condition.

f. The individual tide range annual rates were linearly combined (by assuming that mean tide conditions predominate half the time, and typical neap and spring range conditions each predominate one quarter of the time) to produce average annual net and gross sand transport rates through The inlet.

44. The sand transport calculations used the information above plus a d_{35} grain size of 0.35 mm and Manning's n of 0.035 to produce the sand transport results of Table 5.

45. These results must be understood to be highly approximate (they do not include wave or storm effects, which will be very significant) and also to be only the sand transport *capacity*. For example, even though the 1988 conditions provide a capacity to transport about 5 million cu yd per year in both flood and ebb directions, with a net flood direction transport of 200,000 cu yd per year, the available supply in the bed and from the entrance was much less, so only about 60,000 cu yd per year was been transported into the flood shoal area for that period. (Paragraph 10 cited accumulation of 60,000 cu yd per year on the flood shoal, which will be somewhat less than the net transport, depending on trap efficiency of the shoal.) The results also do not reflect extreme conditions, e.g., the largest tide considered is the average spring range of 4.1 ft.

46. Inspection of the history of depth changes along with the thickness of sand bed in the inlet led to the conclusion that the sand bed within the narrow portion of the inlet was depleted in the mid-1970s. If a supply from the ocean of about 60,000 to 100,000 cu yd per year is assumed, then Table 5 shows why the rate of bed loss continued at a small rate (about 15,000 cu yd per year from 1941 through 1966, as shown in Table 2) until the 1970s, when sand transport began to rise, quickly stripping the bed of sand and exposing the clay beneath. It is probable that, despite the 1988 capacity to transport on the order of 200,000 cu yd per year, the actual rate was near the 60,000 cu yd per year observed to be accumulating on the flood shoal. That amount of

supply would not protect the bed; in fact, it would accelerate the scour of clay by abrading the clay.

47. Table 5 results suggest that for inlet sizes larger than about 30,000 sq ft, only a relatively small quantity of sand would be required to maintain a bed. The inlet could well be stable under the assumed conditions.

Table 5
Sand Transport Rates

Year	Equivalent Area sq ft	Tide Factor C_3	Calculated Sand Transport Rate 1000 cu yd per year	
			Flood Phase	Net
1950	6,700	0.25	1,700	64
1966	9,200	0.40	2,800	110
1974	11,100	0.52	3,500	130
1978	14,500	0.75	4,700	180
1988	18,400	1.0	5,100	200
1988	35,000[1]	1.0	160	7
--	28,500	1.4	2,600	100
--	35,000	1.4	1,100	44
--	45,000	1.5	280	12

[1] Approximately the area of Range 5+00 W.

Clay Erosion

48. Clay erosion calculations were made to examine the recent rates of inlet enlargement and to characterize the bed change for predictions of future evolution. Steps *a* - *c* from the sand transport calculations were again used, except that only the total shear stress was computed, using Manning's equation with n again set to 0.035. For step *d* the clay erosion rates were calculated by the Ariathurai-Partheniades equation (Ariathurai, MacArthur, and Krone 1977),

$$E = M(\tau/\tau_c - 1) \tag{5}$$

where

M = erosion rate constant, g/(sq m - min)
τ = bed shear stress, N/sq m
τ_c = critical shear stress at which erosion begins, N/sq m

Values for M and τ_c are determined by flume tests and vary considerably from one cohesive sediment to another. Therefore, fixing reasonable values a priori is less straightforward than fixing coefficients for sand transport. Ranges of reported values for τ_c are given in Table 6.

Table 6
Range of Values for Erosion Equation Coefficients

Type Clays	τ_c N/sq m
Very soft	2
Soft to medium stiff	7-9
Stiff	20

Values of the erosion rate constant can range from 30 to 300 g/sq m-min, with the higher values applying to stiffer sediments, but without a clear correlation to easily measured parameters (Ariathurai and Arulalandan 1978). In cases where sand is transported over a clay bed, abrasion by the sand grains can cause a somewhat stiff clay to erode as if it were much softer, so that the lower critical shear stress and higher erosion rate constants apply.[1]

49. The computational procedure for steps *e* and *f* was again identical to that described above for sand transport, and clay erosion rates were calculated for neap, mean, and spring tides and then composited for a typical year as described above for sand.

50. The sediment bulk wet density was determined to be 1.70 g/cu cm from lab analysis of core samples--54.9 percent moisture content, 67.4 lb/cu ft dry density (USAED, Philadelphia 1984). Coefficients τ_c and M were varied by trial to generate erosion rates close to the average and maximum rates shown in Table 2, resulting in values of 5.5 to 6.6 *N*/sq m and 35 to 60 gm/sq m-min, respectively. A summary of clay erosion results is given in Table 7.

51. The Table 7 results, which, like the sand transport results, are highly approximate, suggest that the clay erosion will be eventually self-limiting. The clay will not likely become more erodible with depth, and will likely cease eroding altogether at an inlet size of about 26,000 to 30,000 sq ft.

[1] Personal communication, 25 May 1993, Allen M. Teeter, WES, Vicksburg, MS.

Table 7
Clay Erosion Rates, ft per yr

Year	Equivalent Area sq ft	Tide Factor C_3	Average Rate		Maximum Rate	
			Calculated[1]	Observed	Calculated[2]	Observed
1974-1978	11,100-14,500	0.5-0.8	0.3-0.8	1.2	1.9-3.6	2.7
1978-1988	14,500-18,400	0.8-1.0	0.8-0.9	0.8	3.6-4.0	4.0
1988-1992	18,400-18,800	1.0-1.0	0.9-0.8	0.6	4.0-3.9	3.8
--	23,500	1.25	0.4	---	2.2	---
--	26,000	1.33	0.1	---	1.2	---
--	28,500	1.38	0.0	---	0.4	---
--	30,000	1.41	0.0	---	0.1	---
	35,000	1.44	0.0	---	0.0	---

[1] Calculated rate at beginning and end of period with erosion parameters:
τ_c = 6.6 N/sq m and M = 35 g/sq m-min
[2] Calculated rate at beginning and end of period with erosion parameters:
τ_c = 5.5 N/sq m and M = 60 g/sq m-min

4 Causes of Erosion

52. From the above analyses, some tentative conclusions about the causes of Indian River Inlet erosion can be drawn; however, firm conclusions on the quantitative contribution of secondary factors are limited. Three events occurred in the mid-1970s roughly coincident with an acceleration in bed erosion: the sand layer in the inlet was stripped away, exposing the clay layer; the old bridge piles were cut off, probably reducing the inlet's frictional resistance to flow; and the flood tide shoal was mined for sediment to be used in beach nourishment. An argument can be made linking any or all three of these events to the erosion, but the available data do not permit separation of their relative contributions.

Initial Dredging and Jetties

53. Once the inlet size was established with jetties to resist closure, inlet deepening was inevitable. Figure 9 and Table 5 taken together show that the early inlet was on the unstable portion of its response curve. The jetties restricted the supply of sand, the enlarging tidal prism increased the transport rate through the inlet, and scour commenced, pushing the inlet up the response curve. With width constrained, deepening was ordained.

Flow Convergence Zone Scour

54. The deep holes shown in Figure 3 are probably the consequence of local concentration of hydraulic stresses, perhaps at convergence zones of turbulent structures generated by the geometry/roughness or shed by obstructions such as the new bridge piers and remnants of the old bridge.

Scour of Cohesive Sediment Beds

55. The rapid scour since the mid-1970s is at least in part due to exposure of the clay bed when the sand layers were scoured away, and that may in fact be the most significant cause of rapid erosion. Clay eroded from the bed is

immediately transported through the inlet and will not redeposit there in significant quantities; therefore, unlike sand, once clay is entrained, it is lost. The nonhomogeneous nature of the clay beds (deep cores are fairly sparse along the inlet centerline) could be a factor in the development of scour holes much deeper than the average depth, but the flow convergence mentioned above is probably the major contributor to rapid clay erosion.

Old Bridge Removal

56. Removal of the old bridge piers may have reduced the inlet's effective roughness (from a Manning's n of 0.046 to about 0.035) and thus contributed to the growing tidal prism and trend of increasing erosion. However, the evidence is not conclusive, as described above. The old bridge debris (pile stubs and decking) may have *locally* decreased erosion by armoring, causing the ridge observed at Range 1+55 W (see Figure 3) even while contributing to overall erosion by increasing the flow capacity of the inlet.

Interior Shoreline Erosion

57. In 1948 the inlet was 5,500 ft long. Subsequent interior shore erosion opened the landward end of the inlet to where the effective length is now only about 2,000 ft. That shoreline erosion probably contributed to a more rapid increase in tidal prism, and thus more inlet erosion.

Sand Mining

58. Sand mining may have accelerated the erosion somewhat, but the sand transport data of Table 5 suggest it was not a critical factor. The calculations showed a net landward sand transport rate, which suggests that sand deposited on the shoal remained there and did not reenter the inlet under ordinary circumstances. Increased wave penetration may have resulted, leading to more interior shore erosion, which could have been an indirect contribution to inlet erosion.

Sand Bypassing

59. Sand bypassing may have increased the erosion rate somewhat. Given that the sand bypassing is about equal to the net tidal transport in the inlet and less than one fiftieth the gross transport capacity within the inlet, putting 100,000 cu yd of sand per year into the inlet or taking it out does not seem to be a critical factor at this point.

5 Predicting the Probable Equilibrium Inlet Size

Stable Sandy Inlet

60. The Jarrett-O'Brien curve will not serve to predict an Indian River Inlet equilibrium size. A more appropriate calculation is the O'Brien-Dean stability curve in Figure 9, which shows that the inlet was unstable for sizes less than about 20,000 sq ft and *tends* toward stability for cross sections larger than that. Combining that curve with a velocity form of the Jarrett-O'Brien curve suggests that the inlet might stabilize a sand bed at about 34,000 sq ft, *if* the littoral supply were sufficient, but it probably is not, particularly in light of the present sand bypassing. Therefore, neither curve can be used to reliably predict the equilibrium size.

Erosion Resistance of Clay Sediments

61. The clay erosion calculations suggest that erosion will be self-limiting at an inlet size of less than 30,000 sq ft. Spots of continued erosion could still continue. A cross-sectional area of 30,000 sq ft will result from an average depth of 60 ft, which suggests maximum depths of twice that, or 120 ft. However, the Pleistocene sediments at about -100 ft NGVD will probably limit the maximum depth.

Maximum Scour Hole Size

62. The maximum possible scour depth is likely to be about 100 ft--the depth of the Pleistocene sands and gravels. At that depth the sand transport calculations suggest a diminished flow transport capacity meeting a significantly less erodible material. Note, however, that this is not a firm conclusion, since the sand transport calculations assume a near-uniform geometry and do not consider flow convergence zones at all.

63. The lateral size of the scour holes cannot be safely estimated by the simple methods used herein, except in the limit of overall enlargement of the inlet to a nearly uniform depth as described in paragraphs 42-51.

64. The Committee on Tidal Hydraulics notes that experience shows that scour holes in general seem to stabilize at depths of 150 ft or less; very few such holes ever go deeper than that. The evidence for this limiting depth is anecdotal and undocumented, but the observation is supported by several Committee members' experience.

6 Remedial Measures

Architect/Engineer Recommendations

65. The Philadelphia District obtained a consulting firm to identify possible stabilization schemes (Tetra Tech 1992) and received the following suggestions (Committee comments in brackets):

a. Rubble weir at Station 7+00 E (the shallow platform).

[Committee Comment: A single weir is unlikely to impede the flow enough to stabilize or reduce the tidal prism, and it is likely to accelerate erosion elsewhere. The presence of ridges as described in paragraph 16 has not controlled the scour, and may even be a contributing factor to the localized holes as mentioned in paragraph 56. Continued deepening of the adjacent scour holes could cause a weir to collapse.]

b. Rubble weir at Station 0 (the bridge).

[Committee Comment: Unlikely to help for the same reasons as given above. The bridge protection riprap has essentially accomplished this already, though the center portion is lower than the sides.]

c. Armor the entire inlet.

[Committee Comment: It would work, but the enormous cost makes it a last resort. It is probably overkill, in that some lesser solution will probably suffice.]

d. Line the jetty toes with erosion control mats (HYDRACOR mats with frondlike extensions on the surface).

[Committee Comment: Might help, but at present the jetty toes are not threatened. If they are, another approach, such as launching stone techniques (Headquarters, USACE 1991), may also be useful.]

e. A submerged sill with erosion control mats at station 7+00.

[Committee Comment: Unlikely to help. The comments on item *a* also apply to *e*.]

f. Submerged groins at station 7+00 E.

[Committee Comment: Unlikely to help. The comments on item *a* also apply to *f*.]

g. A wood pile weir 250 ft inland from the bridge.

[Committee Comment: Based on the hydraulic calculations here, adding roughness will not stabilize the tidal prism. As stated above, a single weir at any location is not likely to help.]

66. Some combination of these measures, such as weirs spaced at regular intervals, could be effective if designed properly. Extensive design studies would be required for any such plan in order to test its effectiveness and lack of harmful effects. Spaced weirs might be so close together that they approached continuous armoring.

Other Possible Remedial Measures

Additional inlet

67. An additional inlet would contribute part of the tidal prism, perhaps reducing the flow through Indian River Inlet and lessening the erosive pressure. A direct ocean connection to Rehoboth Bay would also contribute to improved circulation and flushing, possibly improving water quality in both bays. (An extremely rigorous design study would be necessary to obtain the potential benefits.) However, cutting a new inlet would further interrupt the littoral sand supply in a region already experiencing beach erosion. Public and political approval of another inlet is unlikely.

Offshore sand mining for inlet nourishment

68. It might be possible to protect the clay bottom from further erosion by maintaining a carpet of sand moving across the inlet bottom. The basic process would consist of occasionally dredging suitable sand from offshore, placing it in the inlet throat, and then recycling it by regularly mining from the flood shoal and reintroducing it to the inlet. A few hundred thousand cubic yards would cover the bottom, but the sand transport rates of Table 5 suggest that several million cubic yards of sand would be required to keep the inlet safely covered. Once the bed was covered, the recycling of sand from the flood shoal would probably have to be at least 200,000 cu yd per year. While this potential solution has several merits, including potentially contributing to

beach nourishment by the amount of sand expelled by each ebb phase, it is unlikely to be an economical solution.

Sand weir in jetties

69. It has been speculated that the ridge observed in Figure 3 at about Range 6+85 E is caused by sand leaking through and over the south jetty and providing a protective layer that inhibits erosion (much like the sand nourishment described above). If that is happening, then assisting the process by creating a path for sand flow at that point could reduce the rate of erosion adjacent to the ridge. Such a path could be created by lowering the jetty to mlw as in weir jetty construction, or by simply removing a few jetty stones to make the jetty more porous. Consideration of such a step will require careful consideration of the possibility that it will exacerbate beach erosion south of the inlet.

7 Conclusions and Recommendations

70. The data and analyses presented here suggest that overall inlet erosion may be slowing. The data do not suggest that deep hole scour is slowing, but it is believed that the deep hole at Range 5+00 W may not deepen further since it has encountered the Pleistocene sediments below the clay.

71. The scour does not yet present a clear danger to the bridge and jetties that justifies remedial measures at this time. Continued regular monitoring is clearly essential, with contingency plans in place to begin action if remedial action is needed.

72. A geotechnical analysis of slope stability to predict at what scour hole size the jetties or bridge will be endangered is recommended. The analysis will provide a criterion against which to compare future scour hole growth and define a decision point for further action.

73. If the Philadelphia District considers restoring the damaged seaward ends of the jetties, streamlined jetty tips to reduce flow convergence should be considered.

74. Continued monitoring should consist of semiannual hydrographic surveys of the bed and of the jetty toes and bridge pier protection for 2 years, then an annual survey thereafter. Survey data should be used to update the types of data presentations used in this report. Biennial surveys of the flood shoal will be helpful.

75. Further studies to prepare for contingencies should include consideration of the following:

a. Borings along the inlet centerline and across Range 7+00 E to define sediment strata down to about 100 ft. Existing geophysical records should be reviewed for adequacy and supplementing, if needed.

b. Erosion testing of the bed clays at several depths to determine the critical shear stress for erosion and the erosion rate constant under flow shear and flow plus sand abrasion. Sample collection and care must carefully conform to laboratory analysis specifications.

c. Measurement of three-dimensional currents with a boat-mounted Acoustic Doppler Current Profiling (ADCP) meter throughout the inlet to supply data for further analysis and to verify a model if needed.

d. A high-resolution, nonhydrostatic, three-dimensional numerical model to map bed stresses, evaluate maximum scour depths, and evaluate remedial measures.[1] A physical model could also be used to define the stresses, but could not adequately reproduce scour and bed change effects on hydrodynamics.

e. Careful evaluation of any proposed remedial measures for other impacts, such as those on shore erosion, water quality, or movement of scour holes to new locations.

Questions Posed by the Philadelphia District

76. The questions posed by the Philadelphia District in paragraph 3 have been implicitly addressed in the preceding sections. Here the Committee's views are explicitly listed.

a. *What factors caused the accelerated scour to begin during the mid-1970s?*

Committee Comment: It is difficult to apportion the responsibility among the several possible contributing causes; however, it is believed that the erosion was inevitable once jetties were constructed. The accelerated scour was most probably caused by exposure of the clay layers to flow convergence stress concentrations, but contributing factors could include removal of the old bridge and flood shoal mining.

(1) *Does sand bypassing affect the scour?*

Committee Comment: The sand bypassing is not significantly affecting the present scour rate since the transport capacity has already stripped the sand bed. At a significantly larger inlet size, the transport capacity may fall to a point that the 100,000 cu yd per year bypassed would otherwise suffice to maintain a stable bed.

(2) *Does the tidal prism-inlet area theory apply?*

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[1] An apparently appropriate model, MAC-3D, has recently been developed by Dr. Bob Bernard of the WES Hydraulics Laboratory. (Personal communication, 25 May 1993, Dr. Bernard).

Committee Comment: The equilibrium tidal prism curve of Jarrett and O'Brien cannot be used to directly infer equilibrium inlet size. In combination with the Dean-O'Brien stability curve, it can be used to qualitatively evaluate inlet stability under constant littoral supply.

b. What course of action is recommended?

Committee Comment: Regular monitoring and preparation of a contingency plan by the path described in paragraphs 68-72.

(1) *Is monitoring alone sufficient at present or should planning-design-budgeting begin?*

Committee Comment: Planning studies providing for the analyses listed in paragraph 71 and 72 should begin.

(2) *What is the threshold for beginning stabilization?*

Committee Comment: Plans for stabilization should proceed if the scour expands its lateral extent toward the bridge or jetty toes, or if the studies described above indicate that scour will continue.

(3) *Are the architect/engineer stabilization study findings appropriate?*

Committee Comment: The Committee does not consider the architect/engineer stabilization schemes as listed to offer a reliable remedy. In particular, single weirs will not be effective. An array of weirs at several of the locations listed might stabilize the inlet, but careful model evaluation will be required.

(4) *Is local cross-section control sufficient?*

Committee Comment: No.

(5) *Should present flows be accepted and the bed armored?*

Committee Comment: Reasonable cost stabilization works are unlikely to alter the flows enough to stabilize the inlet. If stabilization works are needed, some sort of armoring will be the best approach.

c. What modeling tools and/or prototype data are needed to design a stabilization scheme?

Committee Comment: The approach listed earlier in paragraphs 71 and 72 is recommended.

(1) *Should more hydrographic surveys be scheduled?*

Committee Comment: Yes. Survey at least semiannually for 2 years, then annually thereafter.

(2) *What models are needed to evaluate the problem and design stabilization features?*

Committee Comment: See paragraph 72(d).

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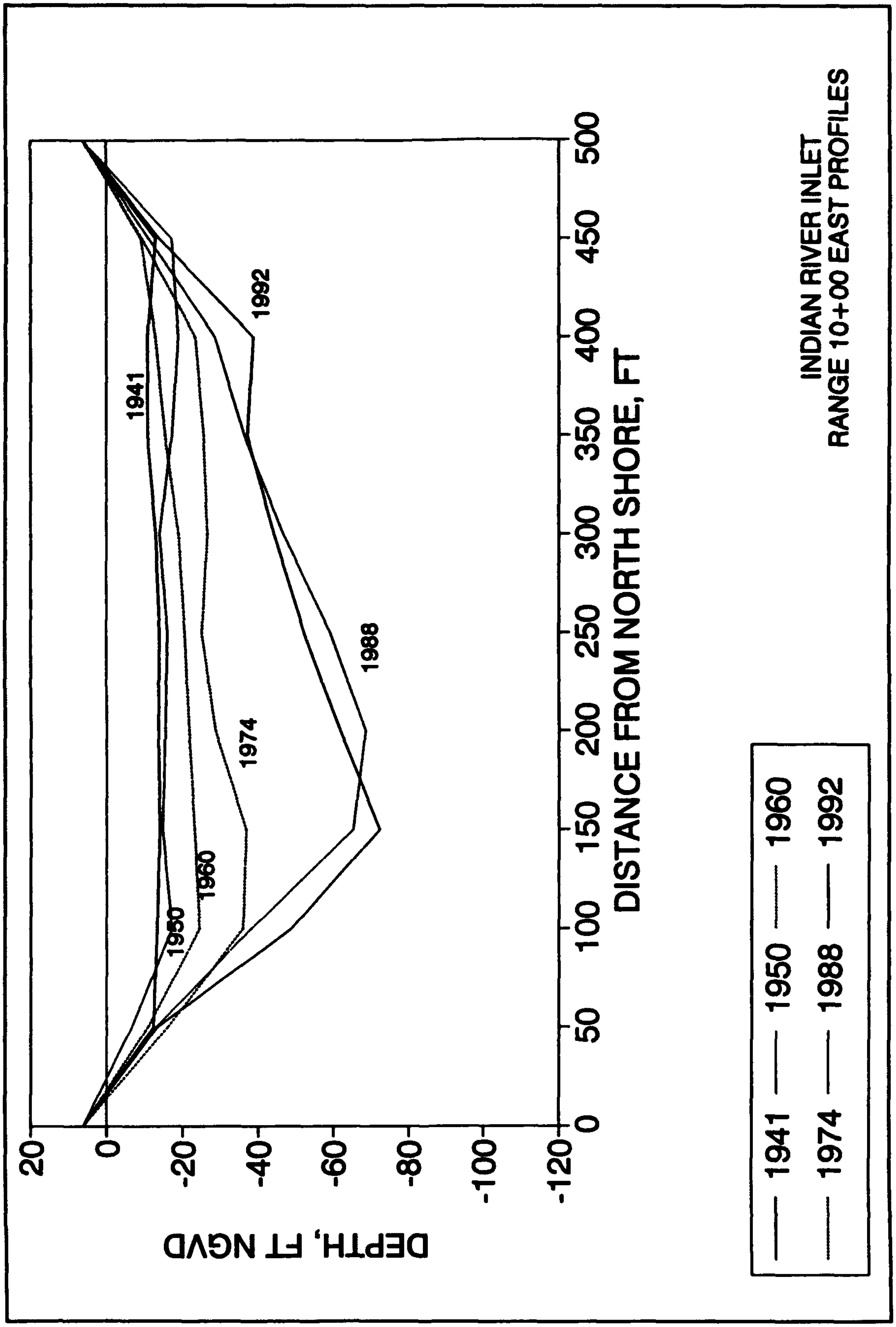
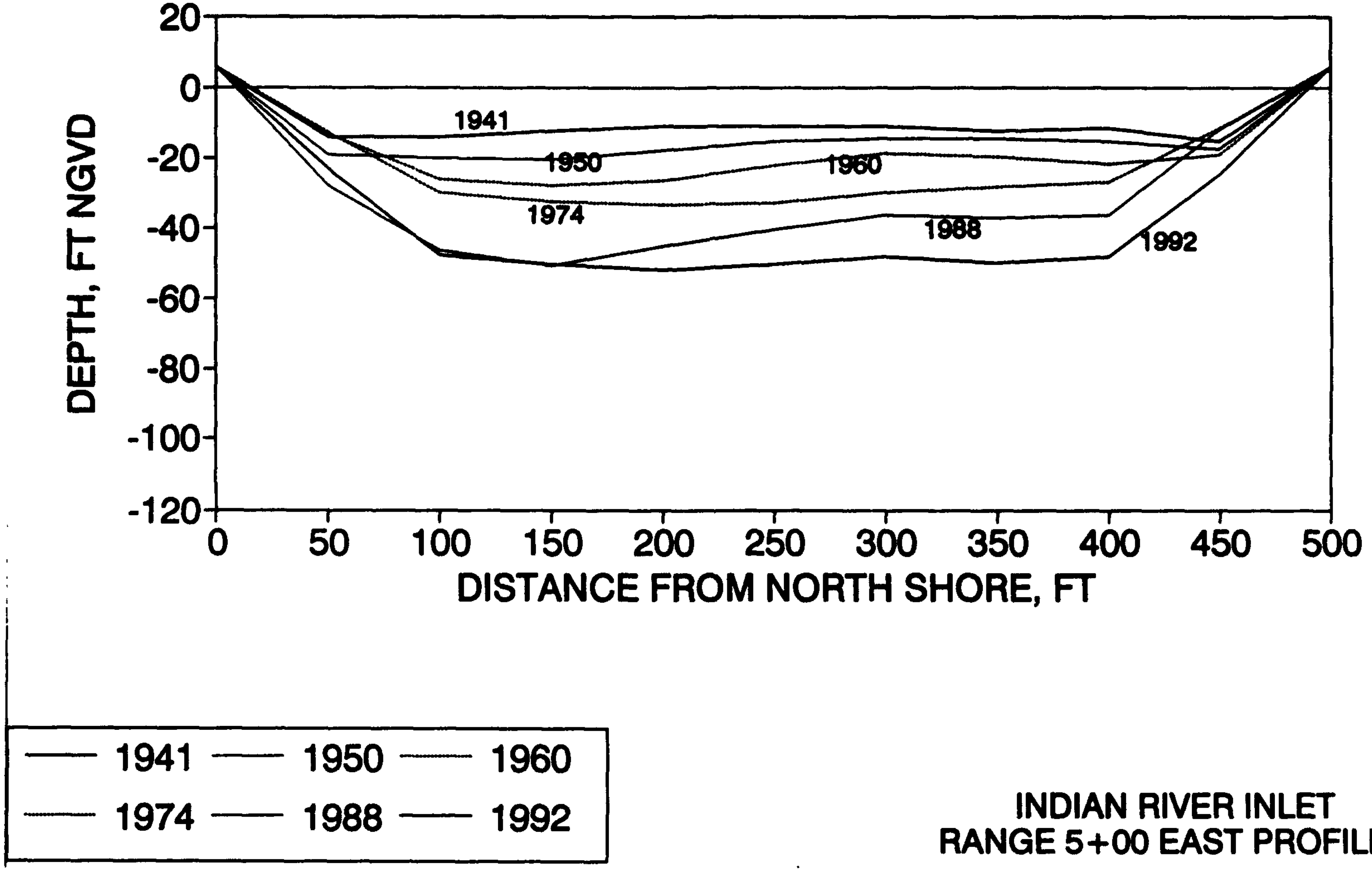
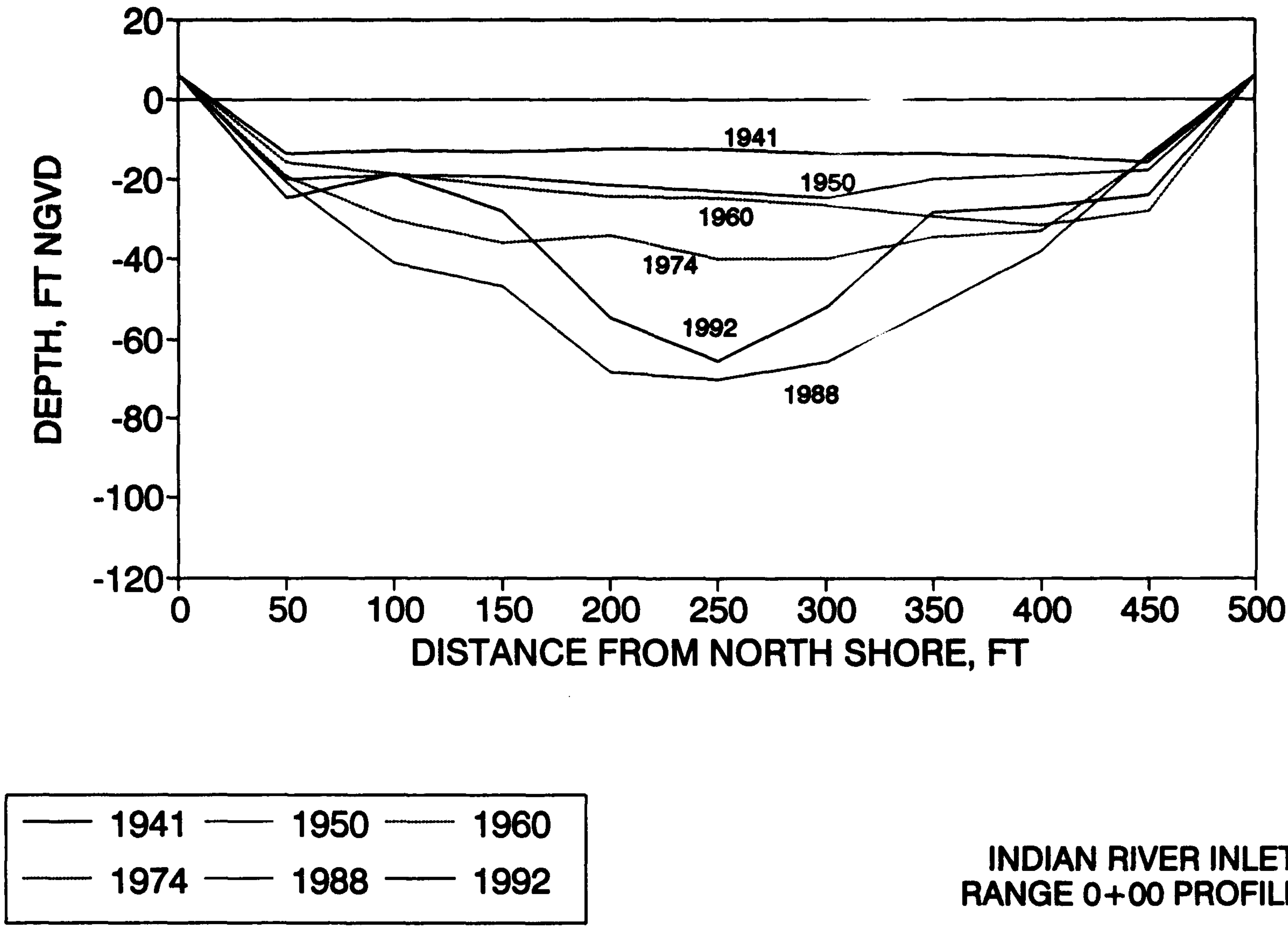


INDIAN RIVER INLET
RANGE 10+00 EAST PROFILES
DEPTH, FT NGVD
DISTANCE FROM NORTH SHORE, FT
20
0
-20
-40
-60
-80
-100
-120
0
50
100
150
200
250
300
350
400
450
500
1941
1950
1960
1974
1988
1992



INDIAN RIVER INLET
RANGE 5+00 EAST PROFILES

Plate 2



INDIAN RIVER INLET
RANGE 0+00 PROFILES

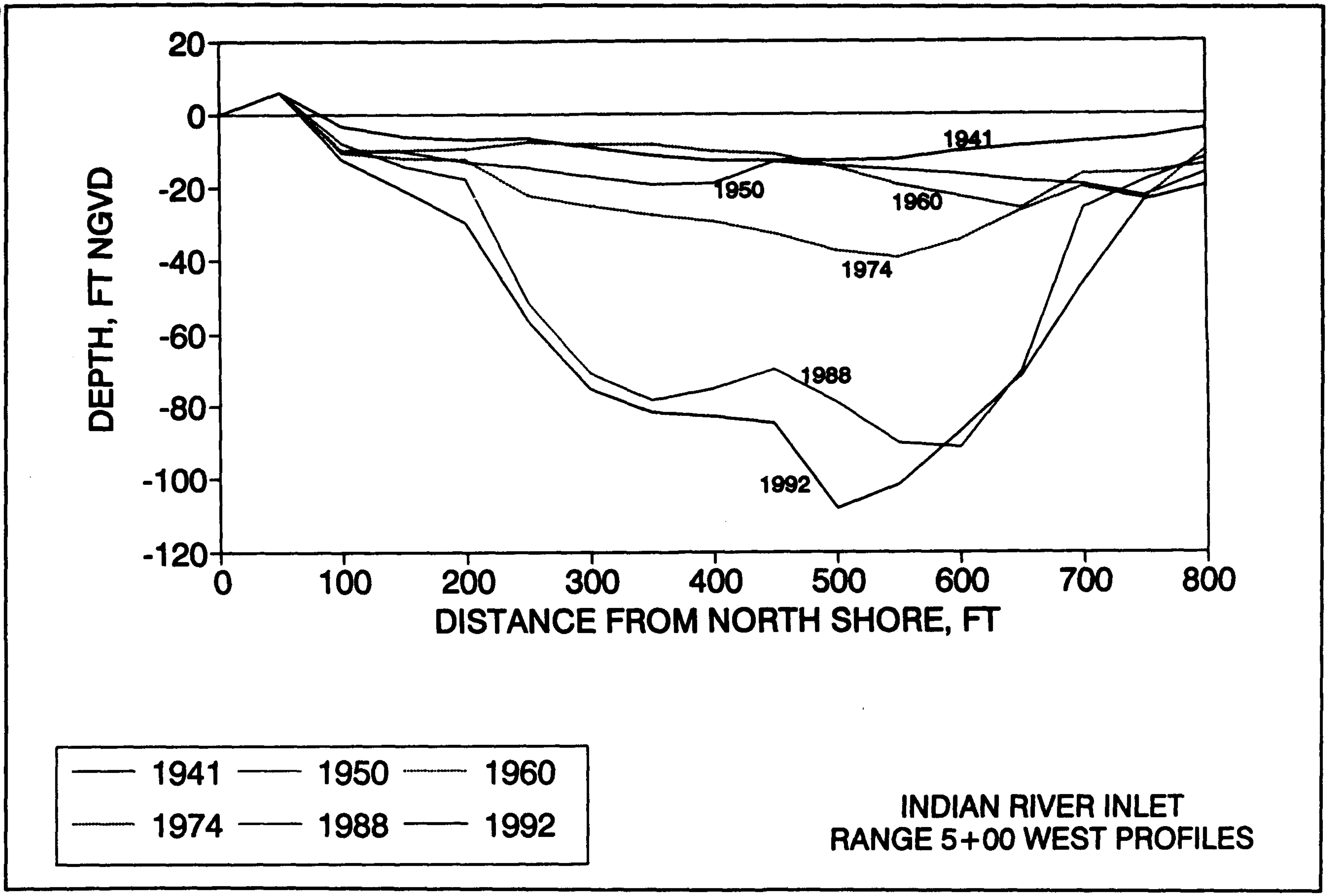
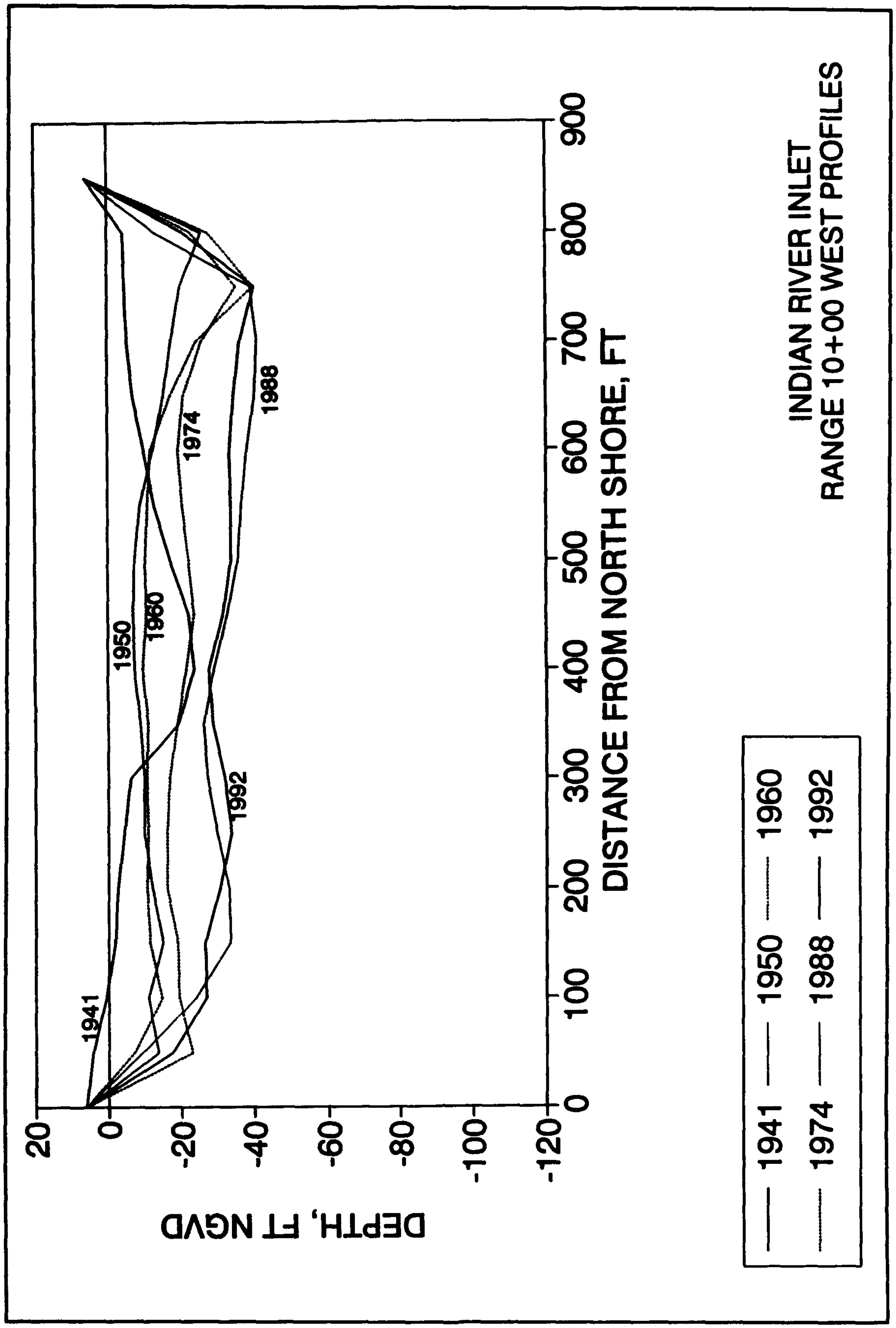
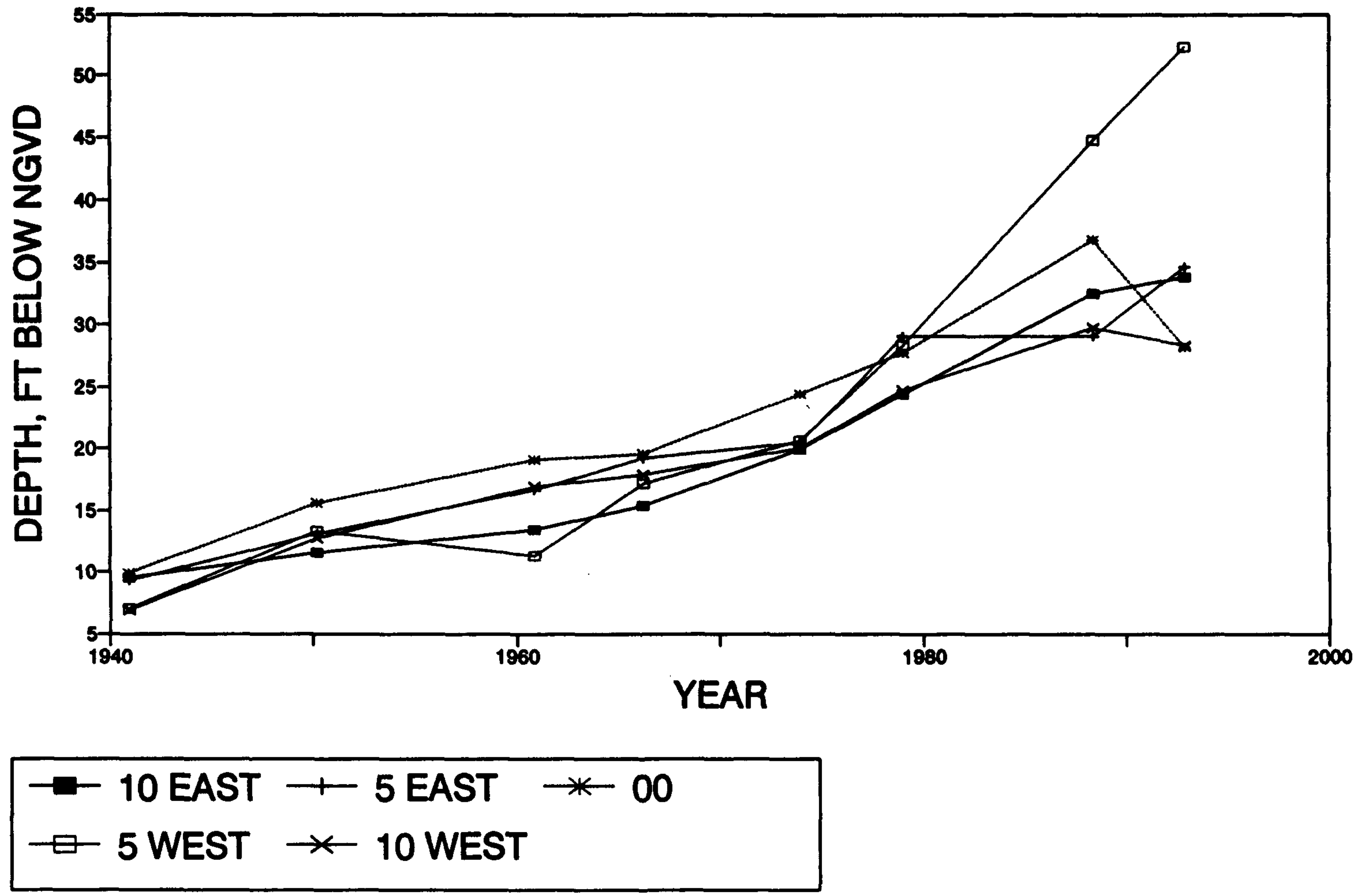

20
0
-20
-40
-60
-80
-100
-120
DEPTH, FT NGVD
0
100
200
300
400
500
600
700
800
DISTANCE FROM NORTH SHORE, FT
1941
1950
1960
1974
1988
1992
1941
1950
1960
1974
1988
1992
INDIAN RIVER INLET
RANGE 5+00 WEST PROFILES

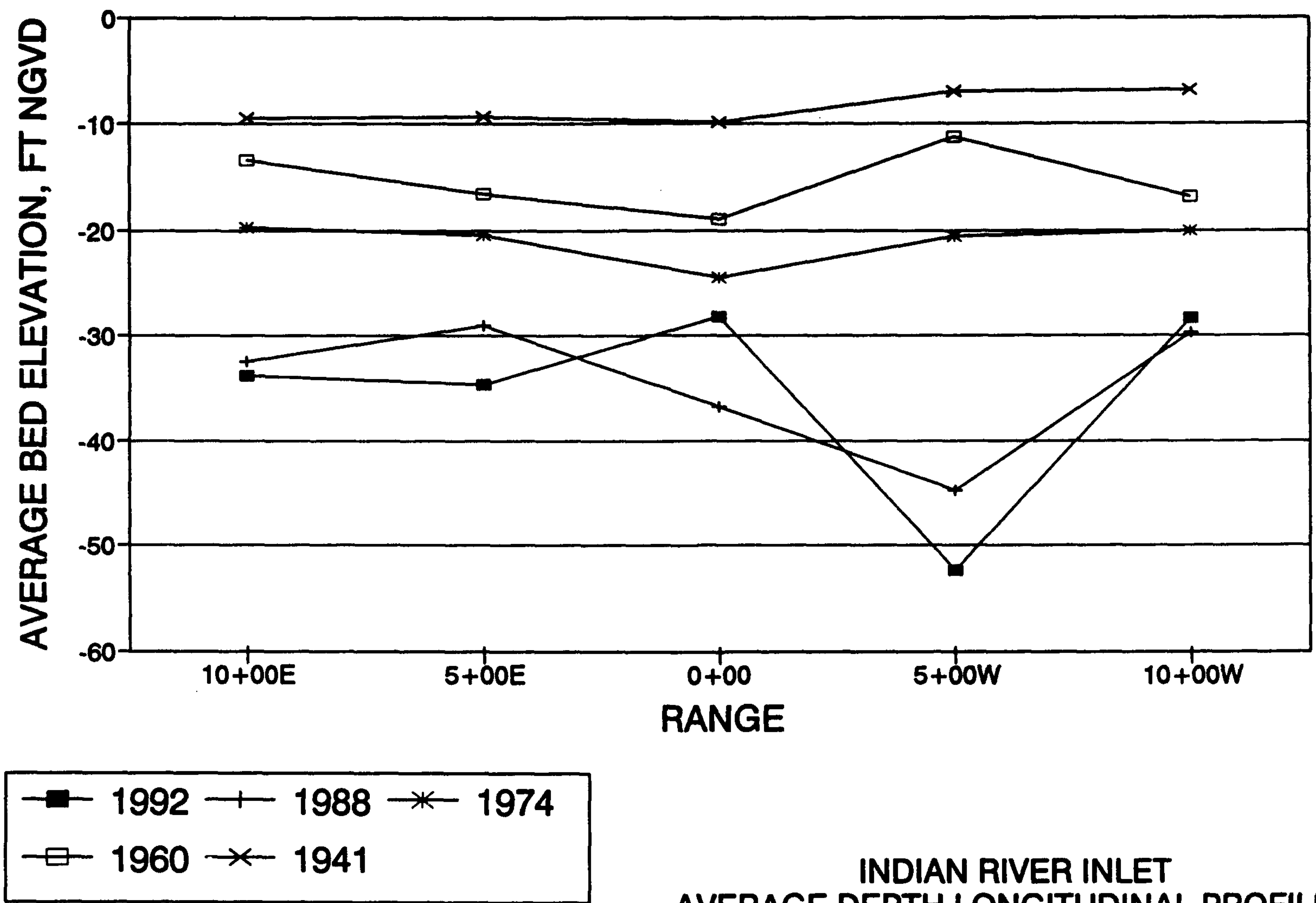
Plate 4


INDIAN RIVER INLET
RANGE 10+00 WEST PROFILES
DEPTH, FT NGVD
DISTANCE FROM NORTH SHORE, FT
20
0
-20
-40
-60
-80
-100
-120
0
100
200
300
400
500
600
700
800
900
1941
1950
1960
1974
1988
1992



INDIAN RIVER INLET
AVERAGE DEPTH BY SECTION, 1941-1992

Plate 6



INDIAN RIVER INLET
AVERAGE DEPTH LONGITUDINAL PROFILES

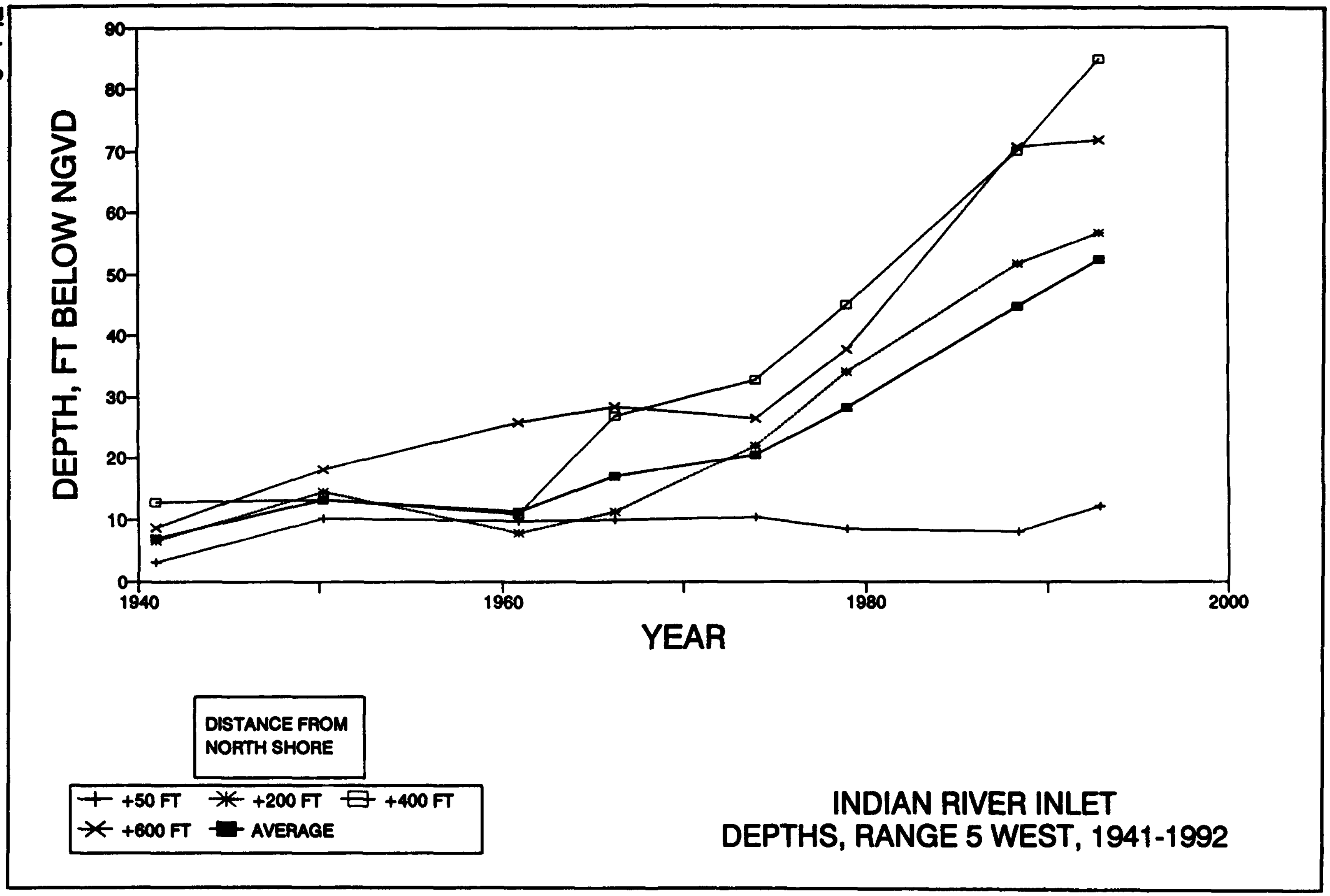

90
80
70
60
50
40
30
20
10
0
DEPTH, FT BELOW NGVD
1940
1960
1980
2000
YEAR
DISTANCE FROM NORTH SHORE
+50 FT
+200 FT
+400 FT
+600 FT
AVERAGE
INDIAN RIVER INLET
DEPTHS, RANGE 5 WEST, 1941-1992

Plate 8

Appendix A
Hydrographic Data

Table A1
Indian River Inlet Hydrography - Range 10+00 East

Distance Across Range ft	Depths, ft Below NGVD, for Indicated Date							
	Nov 92	May 88	Dec 78	Yr 74	Mar 66	Nov 60	Mar 50	Yr 41
0.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
50.0	13.6	13.0	14.9	16.4	10.9	11.2	6.7	12.4
100.0	49.1	38.5	39.2	36.2	10.9	24.8	17.5	13.4
150.0	72.3	65.5	42.6	36.9	10.9	23.4	14.9	14.3
200.0	62.0	68.5	39.6	28.8	10.9	22.0	15.5	14.0
250.0	52.5	59.5	37.4	25.6	10.9	20.8	16.3	14.2
300.0	44.4	47.0	36.8	26.9	10.9	19.0	14.2	13.1
350.0	37.2	36.5	33.9	25.9	10.9	15.5	17.5	10.9
400.0	39.1	29.0	26.5	23.7	10.9	13.3	19.0	10.9
450.0	14.1	11.5	9.9	9.8	10.9	9.4	17.5	13.2
500.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
Avg Depth	33.8	32.5	24.4	19.8	7.8	13.4	11.6	9.5
Area, sq ft	16,922	16,227	12,218	9,918	3,913	6,700	5,777	4,745

Table A2
Indian River Inlet Hydrography - Range 5+00 E

Distance Across Range ft	Depths, ft Below NGVD, for Indicated Date							
	Nov 92	May 88	Dec 78	Yr 74	Mar 66	Nov 60	Mar 50	Yr 41
0.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
50.0	23.1	27.9	29.9	12.9	13.8	13.2	19.4	14.2
100.0	47.5	46.3	40.0	29.9	26.5	26.0	20.3	14.2
150.0	50.2	50.8	47.2	32.4	27.4	27.8	20.5	12.4
200.0	52.0	45.3	48.5	33.2	29.4	26.4	18.0	11.1
250.0	50.4	40.5	46.4	32.9	29.6	22.1	15.3	11.3
300.0	47.9	36.0	43.6	30.0	28.3	18.9	14.5	11.1
350.0	49.7	37.0	33.0	28.0	26.4	19.8	14.7	12.6
400.0	48.3	36.0	28.0	26.9	25.8	21.9	15.4	11.6
450.0	24.6	12.0	14.9	11.4	15.9	19.1	17.5	15.5
500.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
Avg Depth	34.7	29.1	29.0	20.5	19.2	16.7	13.1	9.3
Area, sq ft	17,350	14,536	14,522	10,254	9,595	8,327	6,527	4,636

Table A3
Indian River Inlet Hydrography - Range 0+00

Distance Across Range ft	Depths, ft Below NGVD, for Indicated Date							
	Nov 92	May 88	Dec 78	Yr 74	Mar 66	Nov 60	Mar 50	Yr 41
0.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
50.0	24.5	20.6	25.5	19.3	14.4	15.8	19.9	13.5
100.0	18.5	41.0	30.6	30.2	21.2	18.5	18.9	12.8
150.0	27.9	46.7	41.8	35.9	24.8	21.7	19.1	13.2
200.0	54.6	68.4	46.5	34.0	27.0	24.4	21.5	12.2
250.0	65.7	70.4	47.5	39.7	29.4	24.6	22.7	12.3
300.0	52.1	65.8	44.8	39.9	30.9	26.5	24.5	13.3
350.0	28.2	52.0	35.8	34.3	30.2	29.5	19.9	13.6
400.0	26.9	38.0	32.2	33.1	29.2	31.7	18.9	14.1
450.0	23.8	14.0	12.9	14.5	18.9	27.9	17.9	15.7
500.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
Avg Depth	28.2	36.8	27.8	24.4	19.5	19.0	15.6	9.9
Area, sq ft	14,100	18,404	13,890	12,222	9,727	9,481	7,786	4,940

Table A4
Indian River Inlet Hydrography - Range 5+00 West

Distance Across Range ft	Depths, ft Below NGVD, for Indicated Date							
	Nov 92	May 88	Dec 78	Yr 74	Mar 66	Nov 60	Mar 50	Yr 41
0.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
50.0	12.2	8.0	8.4	10.4	9.9	9.7	10.2	3.2
100.0	20.8	14.1	14.1	12.1	13.0	9.7	10.3	6.4
150.0	29.7	17.9	27.6	12.4	11.9	9.4	13.0	6.9
200.0	56.6	51.6	34.1	22.0	11.3	7.8	14.4	6.6
250.0	75.3	70.9	42.1	25.0	12.8	8.4	16.9	9.3
300.0	81.8	78.5	41.2	27.4	18.4	8.5	19.2	11.2
350.0	83.0	75.4	42.4	29.2	23.9	10.2	18.9	12.7
400.0	84.8	70.0	44.9	32.7	26.9	10.9	13.2	12.7
450.0	108.2	79.1	49.6	37.5	31.9	14.7	14.2	12.6
500.0	101.9	90.1	53.3	39.0	35.9	19.1	15.4	12.2
550.0	86.8	91.3	49.6	34.6	33.4	22.5	16.5	10.2
600.0	71.7	70.5	37.6	26.5	28.4	25.9	18.2	8.6
650.0	46.1	25.8	28.5	20.0	17.9	16.2	19.4	7.5
700.0	23.7	18.0	14.6	23.2	17.6	15.9	22.5	6.6
750.0	19.6	12.0	5.4	9.9	9.4	13.9	15.9	3.9
800.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
Avg Depth	52.4	44.8	28.3	20.6	17.1	11.2	13.3	7.0
Area, sq ft	41,891	35,821	22,654	16,465	13,675	8,978	10,644	5,581

Table A5
Indian River Inlet Hydrography - Range 10+00 West

Distance Across Range ft	Depths, ft Below NGVD, for Indicated Date							
	Nov 92	May 88	Dec 78	Yr 74	Mar 66	Nov 60	Mar 50	Yr 41
0.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
50.0	17.6	9.0	23.9	22.9	11.9	6.9	13.4	-4.4
100.0	27.0	24.0	30.2	19.4	17.4	14.7	10.9	-0.5
150.0	26.5	33.2	26.7	18.9	18.9	11.2	14.9	1.9
200.0	30.4	33.1	21.9	15.9	17.4	10.7	11.9	2.5
250.0	33.6	29.9	20.9	15.9	15.4	10.9	9.9	4.3
300.0	32.0	27.1	23.2	16.7	14.1	10.4	9.9	6.3
350.0	28.6	26.3	26.8	18.9	13.9	10.9	8.9	19.4
400.0	27.5	28.9	31.4	21.5	13.7	9.4	7.4	23.5
450.0	31.6	32.6	33.7	23.7	14.4	10.4	6.9	22.1
500.0	33.9	35.5	34.5	22.3	15.7	10.1	7.4	16.9
550.0	33.6	36.5	33.4	20.2	15.4	10.8	8.9	12.6
600.0	33.5	38.1	29.4	19.1	16.1	11.8	12.4	9.8
650.0	34.6	40.4	28.6	20.7	18.6	16.9	15.4	6.9
700.0	36.4	41.0	32.6	25.9	25.4	24.4	17.4	5.5
750.0	40.4	39.2	31.8	35.2	37.4	39.7	19.9	4.8
800.0	20.7	13.0	13.9	22.8	20.2	27.6	23.9	4.5
850.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
Avg Depth	28.4	29.7	24.8	20.0	17.9	16.9	12.7	6.9
Area, sq ft	24,129	25,255	21,058	17,021	15,172	14,375	10,763	5,843

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13. ABSTRACT (Maximum 200 words)

Indian River Inlet, Delaware, has experienced progressive scour since about 1940, and the scour has accelerated since 1970. The U.S. Army Corps of Engineers Committee on Tidal Hydraulics reviewed the inlet's history, examined its processes, and provided recommendations on needed remedial measures to the U.S. Army Engineer District, Philadelphia.

The inlet, originally less than 20 ft deep, deepened at a rate of about 0.5 ft per year, on average, from 1940 to about 1970. From 1975 on, the rate increased to 1 ft per year, and by 1992 it was deeper than 40 ft everywhere, with some holes more than 100 ft deep. During the period, the north jetty had surffered damage, sand bypassing had begun, and numerous highway bridge piers were removed.

Analyses showed that the inlet was far from an equilibrium size with respect to the O'Brien-Jarrett curve and the balance between sand supply and transport through the inlet. Analysis of sand transport rates and clay erosion rates suggested that a stable inlet might result at an average depth of about 60 ft. The more rapid recent rate of erosion was linked to several potential factors, of which exposure of clay sediment layers is the most probable.

Remedial measures using structures or bed armoring were not recommended at present. Instead, continued monitoring of the inlet and jetties was recommended.

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